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A Severe Tunnel-Lining Fire on the Southern Pacific Railway*

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The Cuesta Grade on the Coast Division of the Southern Pacific Ry. through the Santa Lucia Mountains, 240 miles south of San Francisco, is a maximum 2.20% with a maximum curvature of 10°. The Coast Division is one of the company's two heavy-traffic lines connecting northern and southern California, and is rapidly becoming one of the most popular and fastest passenger divisions in the west. In passing through the range, six tunnels penetrate the mountains, and the road-bed twists around rugged peaks and abrupt cañons; the cañons slightly timbered, and the brushy hills offer fair grazing for stock. The distances between Tunnels 6, 7, 8 and 9 average about 1000 ft.

On Sept. 17, 1913, a tramp prepared breakfast over a small camp fire built in a ditch between Tunnels 7 and 8, and went his way. A mountain breeze fanned the remaining embers into flame, carried the fire to the adjacent grass, rapidly spread over about 1000 acres of mountain slopes, and got to the portal at the east end of Tunnel 7, which was timbered as shown in Fig. 2. The fire resulted in the demoralization of "Coast Line" traffic for 51 days, and made it necessary to handle the congested traffic via the "Valley Route" on a single track over the famous Tehachapi grade.

The fire that eventually destroyed Tunnel 7 was first discovered by Section Foreman Duignan, about 9:30 a.m., Sept. 17, and had then spread over the hills between Tunnel 7 and Tunnel 8. After making an attempt to save the telegraph line over Tunnel 8, and being gradually forced by the heat into the tunnel for refuge, the foreman and his gang started toward Tunnel 7 and discovered that the portal of that tunnel was on fire. Realizing that a passenger train was soon due, Foreman Duignan made a hazardous run through Tunnel 7, which is 1371 ft. long, to get to a telephone booth at Thyle, a spur located just beyond the west portal of Tunnel 7, hoping to communicate with the dispatcher, but he could raise no one eastward on account of wires presumably being down, though he did succeed in raising an agency some 100 miles west. A fire train in charge of a roadmaster and trainmaster with all available forces arrived at the east portal about 3:30 p.m., from San Luis Obispo, 12 miles east. The fire had gotten too great headway by this time, there being a strong wind,

Fig. 1. West End of Tunnel, Looking East, on Oct. 6

(Transfer trail at upper level on left, county road at lower level. Fire had been burning 19 days when this picture was taken.)

and little could be done on account of falling lagging, intense heat, etc. The day was one of the hottest of the season, the temperature being 109° F.

Trains 21 and 22, "The Coasters," had then arrived and a rough trail, approximately 2000 ft. long, was brushed out over the hill and, in spite of the intense heat and steepness of the trail, which was littered with charred brush and hot ashes, all passengers were safely transferred before dark.

The Division Superintendent's special from San Francisco arrived at the fire about midnight and it was decided to seal both ends of the tunnel with timbers and to inject steam in an effort to extinguish the fire. The seals were completed and steam supplied by a locomotive stationed at each end, was turned in about 10 a.m., Sept. 18. The injection of steam was kept up continuously for the next 48 hours.

In the meantime, carpenter gangs assembled and were put to work cutting and framing timbers and preparing to re-timber the tunnel when conditions permitted. All

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trains were detoured, except trains 21 and 22, "The Coasters," and trains 23 and 24, "The Locals," which were allowed to run as far as San Luis Obispo, 12 miles east, and Cuesta, 2 miles west; the passengers and light baggage, mail, and express, being transferred by automobile service over the intervening 14 miles. This method of transfer continued until Oct. 4, at which time a trail 3200 ft. long on an easy grade had been completed around the tunnel and the automobile service discontinued, it being unsatisfactory to railroad company and passengers on account of the dangerous mountain road. The trail proved enjoyable and satisfactory to passengers and some 6000 people traveled over it in the intervening month before the tunnel was again opened to traffic.

Early in the morning of Sept. 20, small cracks were appearing above and back of the seals and it was discovered that the steam, to a small extent, was penetrating and dissolving the rock, which is of an argillaceous formation. Small cave-ins occurred and it was thought advisable to discontinue the pumping of steam.

Arrangements had previously been made with The Pure Carbonic Co., of Berkeley, Calif., for a carload of 99% pure carbon dioxide and, upon its arrival the forenoon of the 20th, the plant was set up at the west end and the liberation of the gas into the tunnel began at 6 p.m. Applications were made at intervals until 5 o'clock the following morning, about 10,400 lb. of gas being used. The gas was shipped in steel containers into which it had been forced under pressure of 1000 lb. per sq.in. at a temperature of 60° above zero. The containers were of two sizes; the smaller containing 22 lb. or 170 cu.ft.; the larger containing 52 lb. or 400 cu.ft. When liberated, the gas expands at the ratio of 55 to 1 and the temperature drops to 104° below zero. The chemist accompanying the shipment, directed his efforts toward getting a test of 10% of gas at the east or lower end of the tunnel.

The method of injecting the gas consisted of assembling 14 containers and connecting them to one pipe, the group being designated a "battery." This battery was set up about 86 ft. from the west portal on a level with the track and connected up to one 2-in pipe through which was threaded a 1-in. steam line for warming the gas. This double pipe projected into the tunnel for a distance of about 15 ft. beyond the seal. Seven containers were in constant discharge, the switch being made to the opposite seven when the first seven became exhausted. Ordinarily, it required 18 min. to discharge a battery of 14 containers.

A thermometer inserted into the tunnel through a porthole in the seal on the east end about 9 o'clock registered 212° F. The test for gas at this time showed the presence of 5.5%. At 2:50 a.m., Sept. 22, following a small cave-in back of the seal on the east end, the gas test showed 5.2%, the temperature averaging 200°. The temperature at this end had dropped to 185° by 7 o'clock.

Cave-ins had continued to occur back of both seals and it was decided to open up both ends. Immediately after the seal was removed from the west end about 8 o'clock, Sept. 22, a string of four flat cars was pushed to the face of the tunnel and 50 men started removing about 200 cu.yd. of material which had caved in with the removal of the seal. Ten men were installed at the face and worked 10-min. shifts, passing the dirt to the first car, from which it was worked by a series of passes to the last car. The volume of smoke and gas, which was small at first, gradually increased and at 10 o'clock it was generally conceded that fire still existed in the tunnel, possibly piles of embers that the gas had not penetrated.

After removing the seals there was no timber in sight in the east end, nothing remaining but piles of charcoal and ashes. On the west end, however, the timbers were standing intact as far as could be seen. Several trips of inspection were made by volunteers, in each case the men returning exhausted and nearly overcome by the heat and gas. The fact, however, was determined that about 400 ft. of the tunnel timbers were unburnt but that fire was still alive beyond this point. The tunnel was then resealed and another consignment of carbon dioxide ordered.

The second experiment with the gas began at 3 o'clock in the morning of Sept. 23. Thermometer readings taken just before the gas was turned in showed the temperature to be 194° at the west end and 302° at the east end. Temperatures taken and tests for gas made at frequent intervals during the 28 hr. of the second application showed that, as the percentage of gas increased, the temperature dropped. The temperature on the west end at 7 a.m., Sept. 25, registered 113°, the gas test recording 16.8%. The maximum gas test during the experiment was 25.2%. On the east end, the temperature fluctuated between 212° and 140°, the gas test showing a maximum of 15%. In the second application 13,032 lb. of gas were used.

It was then decided to remove the seal on the west end only, the east end to remain closed. Accordingly, the west seal was removed at 8 a.m. and men worked their way into the tunnel under protection of a battery of air nozzles fed by four air pumps through ordinary fire hose. An attempt was made to erect a bulkhead of 1-in. lumber between posts at a point 216 ft. in but men could not stand the heat and gas and the bulkhead was abandoned after being built to the springing line. A second attempt was made to erect a bulkhead at a point 200 ft. in, this time the bulkhead being assembled on the outside and carried in in sections. This was also abandoned when about half done on account of heat and gas, the latter being so strong that it was necessary to use fusees for lights, a fusee being the only kind of light that would burn. The heat at this time registered 96° at the floor and much hotter at the roof. Several of the strongest men who volunteered were overcome, requiring assistance to the outside and resuscitation. A third attempt was made to erect a bulkhead at a point 60 ft. in and abandoned when half complete for the same reasons as above stated.

While the attempt was being made to erect a bulkhead in the west end, it became apparent that some scheme should be devised whereby the gas and heat could be forced out of the tunnel without generating sufficient draft to rekindle the fire. Accordingly, a wooden chimney, 3 ft. by 4 ft. by 24 ft. long, was constructed and placed upright with its base just above the roof line of the tunnel on the east end. A pipe was connected at the bottom and steam forced upward through the chimney beginning at 7:40 p.m. This forced draft drew great quantities of smoke and gas through the chimney.

On the west end, the draft was established westward and a stream of steam and smoke about 1 ft. thick moved along just below the top segments, increasing in an hour to a thickness of about 2 ft. About 8:30 p.m.,

an inspection was made in the west end and it was found that the atmosphere was clear and cool, the "sucker" seeming to have the effect of lifting all gas and heat from the tunnel floor. At 9 o'clock the steam in the chimney was shut off and a porthole about 30 in. square made in the east seal. Through the hole it was possible to see about 60 ft. into the tunnel, the atmosphere being fairly clear. The presence, also, of charcoal flames was discovered on the floor and the porthole was immediately resealed. For the remainder of the night a small quantity of steam and light smoke escaped from the west portal.

The next morning, Sept. 26, the temperature on the west end had dropped to 70° at the floor line and 104° at the roof. It was now possible to complete the bulkhead at the 60-ft. point which had been abandoned the night before. Temperature in the east end ranged from 178° to 242°, the highest reading being obtained by resting the thermometer on the rock at the roof line. The seal was pulled out of the east end about 9 a.m., very little gas or smoke escaping, and gangs put to work mucking out preparatory to setting a steam shovel in. At 8 o'clock that night the temperature at a point 24 ft. in, registered 311°.

In the west end, the bore seemed to be gradually cooling and it was thought that, on the morrow, it would be sufficiently cool for men to enter and complete the bulkhead at a point 200 ft. in and, from this bulkhead, examinations could be made, by men in oxygen helmets, of a large part of the unexplored section of the tunnel. It was arranged to borrow a couple of experienced firemen from the San Francisco Fire Department with their oxygen helmets. During the day, several jars or quakes were felt by men working in the vicinity of the east portal and by night it was suspicioned that heavy masses of rock were falling from the roof and closing the bore. It was later discovered that this suspicion was well founded.

On the morning of Sept. 27, a 5-yd. steam shovel started on the east end to remove the material which had slid in when the portal burned. It was necessary to handle the shovel with a work engine, the engine alternately pushing the shovel to the slide and pulling it out to dump into a ravine about 60 ft. from the portal. A continuous jack-arm rest was made by using three rails, the middle one inverted, interlocked and supported by blocking. Early in the afternoon the heat became very intense on the east end and it was feared that the fact of that end being open was causing sufficient draft to rekindle fire. A wooden curtain was constructed over the hole and it was found necessary to continually play water on it to prevent its burning.

The two San Francisco firemen arrived at the west end about 4 p.m., Sept. 27, and up to 9 o'clock had, by successive trips, worked their way into the tunnel a distance of 320 ft., the temperature being 165° beyond the 216-ft. bulkhead previously mentioned. On their last trip in, the fire could be seen at a point about 30 ft. beyond and it was suggested that men go in with water lines, protected by air nozzles, and attempt to extinguish the flames. Following this suggestion, a bulkhead was constructed on a push car and about 10 o'clock a charge was made on the three bulkheads, 60 ft., 200 ft. and 216 ft. in, respectively, knocking them down and the portable bulkhead rolled ahead for protection against heat and smoke. At a point 280 ft. in, a small pile of débris was encountered which, after being well soaked, was shoveled to the sides in order to crowd the bulkhead forward. The débris consisted of smoldering segments, lagging, muck, etc. At this time the atmosphere was quite clear in the immediate vicinity of the air blower and water line. The smoke, gathering in a dense cloud along the roof, made it impossible to see what condition the timbers above were in and, as advance was made, the force of the water was played on the roof to dislodge any loose lagging or rocks, thereby making it safer to pass under.

At a point 330 ft. in, a small pile of smoldering embers blocked the passage of the car and the packing was on fire at this point. By 11 o'clock this fire had presumably been extinguished and the car with bulkhead advanced to a point 380 ft. in, where another pile of smoldering débris blocked the passage and it was discovered that there was considerable fire in the overhead packing at this point. Ahead, for an indefinite distance, could be seen a bright illumination. As advance was made, the heat became more intense and the smoke hovered nearer the track, making it very difficult and discouraging for men to come to the front to relieve those handling the nozzle. It became necessary by this time to relieve men at 5-min. intervals.

The playing of water on the débris at and beyond the point 380 ft. in continued until about 1:30 a.m., Sept. 28, when, suddenly, the top timbers 50 ft. behind burst into flame and the men on the nozzle had to make a hasty and perilous retreat through the falling embers, some of the men requiring assistance to the outside and resuscitation. The water and air lines were quickly withdrawn and a vain attempt made to confine the fire. The heat and smoke, however, was too great and at 4 a.m., all efforts were centered on protecting the portal. It was then realized that the only thing to do was to let the timbers burn out as quickly as possible. The slipping condition of the ground around the portal made it imperative to keep the timbers standing so as to prevent the possibility of a slide coming down and bottling the fire. By 6 o'clock, the portal was aflame and slowly crumbled away. At 3 p.m., the bore was closed except for a small hole at the top through which escaped heavy black smoke and steam.

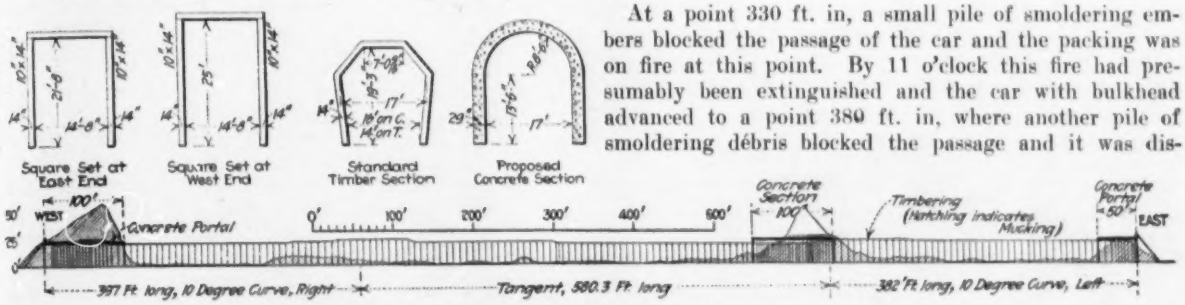

FIG. 2. TUNNEL NO. 7, COAST DIVISION, SOUTHERN PACIFIC CO.

(Original timbering, shown above as "Standard Timber Section," 10x14-in. sticks with 3-in. plank lagging throughout. Permanent retimbering of same design, but temporary retimbering in square sets as shown. All sets spaced 2½ ft. c. to c.)

The steam shovel on the east end had cleaned out all of the material it could reach by 4 p.m., Sept. 28, and was sent away, the balance of the mucking being done by hand. On Sept. 29, the temperature had dropped sufficiently to permit of an inspection being made and, although the floor was covered with piles of hot embers and rock and an intimidating rock would occasionally fall from the roof, the fact was determined that for 450 ft., there were no timbers and at this point a slide had come in from the roof almost completely filling the bore. This proved, later, to be a cave which entirely blocked the tunnel and extended up into the mountain an indeterminate distance. The rock in the cave was of a serpentine formation and, subjected to the intense heat, had crumbled somewhat, although some rocks of a diabase formation were also found.

In the meantime, in the east end, a few temporary portal timbers had been erected and the track gangs employed in cleaning out muck, taking out distorted rail, and laying new track. This was tedious and dangerous work, the temperature ranging as high as 160° for several days and not until an air blower was installed was there any decided lowering of temperature. Water was used extensively to cool off the floor but the too rapid cooling made the danger from spalling rocks greater. Spalls and rocks to an average depth of three feet covered the track. Carbide lights were used at first but these added to the heat, and the danger from explosion was ever present because of so few men understanding their use. Electric lights were later installed. The progress for the first 400 ft. into the tunnel was very good but, on reaching the caved-in material, became very slow and tedious. The gas was bad and the temperature ranged between 80° on the floor and 125° at the roof.

While work was thus progressing on the east end, the steam shovel at the west end had cleaned out the cave at the portal and the rock had cooled sufficiently to permit the setting of temporary portal timbers. Beyond the portal the temperature of the rock in the roof ranged as high as 274°. On one occasion the temporary portal timbers caught fire. Water was used in this end, also, for cooling purposes and an air blower installed. On Oct. 11, fifteen sets of posts were placed on 8-ft. centers but not until the 15th could the segments be raised.

The débris on the track in this end was very light and no amount was encountered until reaching a point about 250 ft. in, and from this point to the toe of the slide piles about 5 ft. high alternated with almost clear spaces. Work progressed favorably for 800 ft., which brought the temporary timbering to the toe of the cave-in. At this time the slide was estimated to be 60 ft. through along the roof line and 110 ft. at the bottom (see Fig. 2).

On the morning of Oct. 17, in the east end, the timbers having been set for 393 ft. in, the ground began to work badly and a big slide took place, knocking out one set of timbers and filling up about 24 ft. of the tunnel. All ideas of mucking out the slide by ordinary methods were then abandoned and it was decided to drive a 6x6-ft. drift high enough in the cave so that sets of long timbers could be placed, within which a concrete section could be later built. Seven of these sets were placed on 3-ft. centers with great difficulty, the material continually crowding down and finally they were abandoned for the lower sets. This drift, which was 52 ft. long, was driven in exactly three days, the miners suffering much from the heat which at times ranged as high as 147°. In addition to the heat there was considerable charcoal gas and steam. Upon excavating, later, it was found that a great deal of semi-burnt smoldering timber was buried in the material from which the foul gases emanated. As soon as the drift was completed, breast boards were driven and excavation and erection of timbers followed shortly.

On the west end, timbering was progressing rapidly and by Oct. 25, was within 46 ft. of the drift. Progress then began to drag on account of heavy ground encountered in the east end and the amount of muck increasing in the west end, the tunnel floor being covered to a depth of about 8 ft. with huge rocks which had to be broken up with gads and hammers. At this stage, to insure safety to the men working ahead of the timbered section, five 12x12-in.x38-ft. crown bars were placed, spanning the gap between the last set of timbers on the west end and the crown of the cave.

Both ends were now being worked under almost identical conditions. At noon on Oct. 30, the timbers in the drift showed signs of failure and by 9 p.m. it was thought advisable to place two 12x14-in.x32-ft. timbers

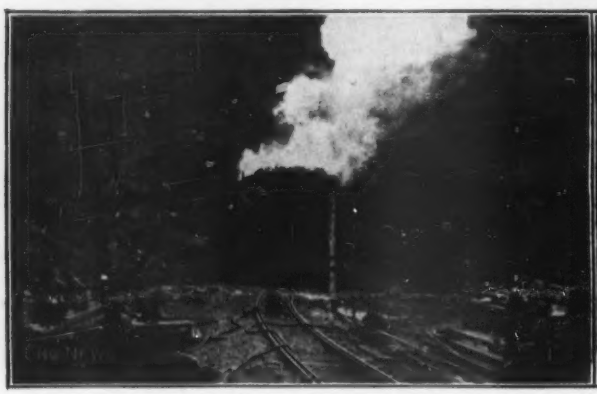

FIG. 3. WEST PORTAL ON SEPT. 28

(This picture was taken early in the morning following a strenuous night of fire fighting. Flames were flickering over the top timbers of the portal.)

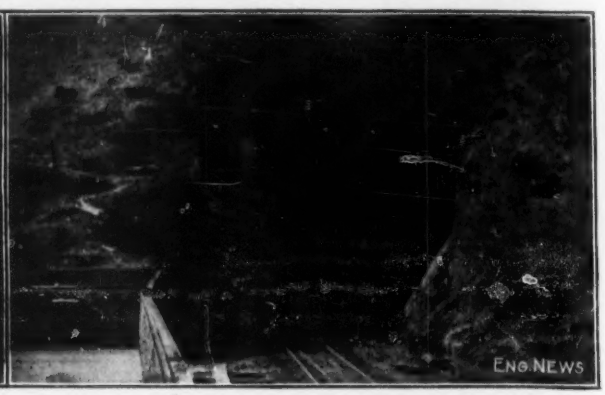

FIG. 4. WEST PORTAL ON OCT. 7, SHOWING CAVE

(After steam shovel had removed all the material it could reach. The distance from top of rail to top of slide is 65 ft. The steam shovel worked during the last few hours under an overhang of about 10 ft. and to protect the craneman from falling rocks, it was necessary to construct a timber canopy on the boom.)

through the drift to reinforce it. By the evening of Nov. 2, the gap between the two ends of the tunnel had been reduced to 28 ft. and mucking was progressing slowly but favorably.

The next 24 hr. saw the gap reduced to 11 ft. and it was decided to take this material all out at one time. Accordingly, five 12x12-in. crown bars were driven above the original drift, the ends made to rest on the timbers already in place. With this protection, the balance of the muck was removed, timbers put in place, lagged, and at 3 a.m., Nov. 4, a work train was run through the tunnel. Normal traffic was resumed on Nov. 7.

In the course of retimbering, a total of 11,780 cu.yd. of rock and earth, which had dropped from the walls and roof of the tunnel, was removed. Of this amount, 2890 yd. were from the cave-in. All temporary timbers, except those at the cave-in, were placed on 8-ft. centers. At one time 350 men were employed, the average being 250 men per day working 8-hr. shifts.

To furnish steam for the generators supplying electricity for lighting purposes and to operate the water pumps, a switch engine was spurred out at each end of the tunnel and used as a stationary. Gasoline engines were used to operate the pumps which supplied the 12-in. air lines. A complete telephone system was installed, the east-end station being designated as "Hurry-up"; the west end being known as "Hows-she-coming." A doctor was in constant attendance and the camp boasted of a drug store, hospital, commissary, a scenic boulevard, freight yards, and one of the largest fire departments in the state.

Owing to the roughness of the country, it was necessary to spur out the east-end outfits at a point 4000 ft. from the tunnel and at the west end the outfits were spurred out 6000 ft. from the tunnel, the men being hauled to and from work by the work train. Short spurs were laid at each end for switching facilities. Switching was quite difficult on account of the 2.2% grade and 10° curves.

Concrete work is now in progress and it is proposed to construct concrete portals, a 50-ft. concrete section in the east end, a 100-ft. section in the west end, and a 100-ft. section at the point where the large cave-in occurred.

Cost Data

An absolutely accurate detailed cost statement is not available at the present time. However, the following figures are a very close approximation:

	Labor	Material	Total
Extinguishing fire	$5,800		$5,800
Relaying track	1,500	$1,300	2,800
Mucking, including disposition of same	23,000		23,000
Temporary tracks for outfit cars	1,500	2,500	4,000
Handling material	1,500		1,500
Framing and placing timbers, 825,000 ft. b.m.	11,000	15,500	26,500
Steam, air and water lines, laying and operating	4,000	1,000	5,000
Electric lights, installing and operating	700	250	950
Work-train service, including fuel	7,500	3,500	11,000
Fuel, gasoline, candles, etc.		1,700	1,700
Hardware, bolts, spikes, etc.		400	400
Miscellaneous, hose, sacks, canvas, carbonic gas, old ties, wedges, etc.		4,000	4,000
Shop charges, store expense, use of tools, etc.		2,000	2,000
Building trails, handling baggage, mail, express etc.	1,200		1,200
Automobile hire, transferring	6,000		6,000
Incidentals, cleaning up, etc	300		300
Total	$64,000	$32,150	$96,150

Of this amount, it is expected that about $70,000 will be handled by the insurance fund. The concrete work, which is now in progress, is being handled as an "Addition and Betterment."

The work was carried out by the forces of the Southern Pacific Co., under the direction of Thomas Ahern, Division Superintendent and R. M. Drake, District Engineer. The writers were on the work practically all of the time.

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A Very Heavy Test Load on a 40-ft. reinforced-concrete highway bridge was made in 1911-12 by the Illinois Highway Commission under the direction of A. N. Johnson, the engineer for the Commission, and is described in a paper by D. A. Abrams in the 1913 "Proceedings" of the American Society for Testing Materials, just issued. The bridge under test was a 40-ft. reinforced-concrete girder bridge, in which the two outside girders and floor were of monolithic construction. It was designed for the usual loading of 125 lb. per sq.ft. uniform load or a 24-ton traction engine load distributed over an area 10x12 ft. It was completed about Jan. 1, 1908, and was tested for small loads several times previous to July, 1911, when the load here noted was placed and continued from that time until September, 1912. Deflection and deformation measurements were made at different times during the entire loading. The maximum load was equivalent to 309 tons per girder, in addition to the weight of the bridge, and the floor slab carried an applied load of 418 tons for over a year without signs of serious stress. These loads amount from nine to thirteen times the maximum load that a bridge of this kind would be expected to carry in service. The maximum stress of the longitudinal girder steel as determined from the deformation measurement was about 25,000 lb. per sq.in. The stress computed for the same load under the assumption that steel for the girders takes all the tensile stress, is 47,000 lb. per sq.in. It is rather difficult with the irregular section of the bridge to determine the exact position of the neutral axis from theoretical considerations, but it is evident from the difference between the two figures given above that the concrete must have carried in tension a considerable part of the stress. The paper by Mr. Abrams has been reprinted in separate form. It contains drawings of the bridge, views of it during construction, and all of the resulting figures of the computations.

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Concrete Tile Blocks are being manufactured in a specially built factory in the Canal Zone, for use of the new Administration Building at Balboa. The process used is the invention of A. A. Pauley, who installed the manufacturing plant and who has granted the sole right for manufacture on the Isthmus of Panama to the United States. The blocks are manufactured by a wet process quite different from the dry process generally used in making concrete blocks.

Concrete is mixed in the plant in the proportion of one part of cement, one and one-half parts of sand, and two parts of stone screenings. It is mixed fairly wet. Each molding machine is capable of turning out four of the smaller blocks at a single operation, and three of the larger sized blocks. The forms are inclosed within a double cast-steel lining with a space between for the priming steam. In molding a block the first step is to insert at the bottom of the form a cast-iron frame having the shape of the solid portion of the required block. This acts as a base and support for the block. The valve from the concrete mixer is then opened and the concrete is allowed to flow down a short chute to the top of the mold. A workman then diverts the thin or wet concrete with a trowel into the crevices around the forms until all are filled level with the top. Steam, which may be graduated to the desired pressure by a series of valves located beneath the machine, is then turned on for a period of four minutes, at the end of which time the concrete has so stiffened that it will stand alone and is turned out of the mold by means of a crank, which pushes out the interior form. The blocks are then lifted from the mold and transferred to short lengths of wooden planking and placed on receiving racks, which are on a combined moving platform and elevator system, which carries the blocks to the curing room.

The curing room consists of 24 galvanized-iron compartments, each about 45 ft. long, 4 ft. wide, and 8 ft. high, entirely inclosed. In these rooms the blocks are given their perfecting treatment, which consists of a combined steam and water dripping process, devised to facilitate the setting up and hardening of the blocks, so that within 48 hours they are ready to be removed to the storage yard.

Each unit of the plant is operated by individual electric motors. The steam is generated by two boilers located in a shed outside the plant.

Blocks to be made are 12x12x18 in., for foundations or walls, 8x12x12 in. for main walls, 8x8x16 in. for corners, 6x12x12 in. and 4x12x12 in. for partitions, and 3x12x12 in. for interior columns.

Survey in the Sonora Desert

By Otto Lemberger*

SYNOPSIS—A narrative account of a survey in an adjacent but little known part of the world, the western part of the Sonora Desert, in Mexico, south of the State of Arizona, U. S. A.; giving an interesting account of the difficulties, hardships and adventures of a surveyor, as commander-in-chief of a hastily recruited party; illustrating the engineer in some of his most fascinating work —a pathfinder and explorer in the desert and the harbinger of civilization—and a description of the country and its possibilities.

One day in August, 1910, two gentlemen entered my office in Chihuahua, State of Chihuahua, Mexico, and asked if I could survey for them a tract of approximately 300,000 acres of mesa land in the little known Sonora desert, which is a continuation in Mexico of the ill-famed Yuma desert of Arizona. They showed me a map of the land and described the country as a mesa devoid of vegetation and level as a table. According to their opinion there was not the least obstacle to overcome. We agreed upon a salary and expenses, to begin from the day I left Chihuahua until my return to that city. Work was expected to start in November.

Fig. 1. Survey Party on the March

November came but with it a revolution in the State of Chihuahua, insignificant for several months, but ending eventually in the conflagration which overthrew the Diaz government. It was not until after many postponements and disappointments that a permit was finally granted by the Madero government for the survey, to be made in four months dating from Oct. 24, 1911. On Dec. 2, I left Chihuahua for Yuma, Ariz., to take up the work.

Although I had been repeatedly informed by the promoters that preparations for the survey would be made before my arrival at Yuma, so that I could devote my whole time to the survey work, it was ten days before I was able to leave Yuma with a party hastily recruited.

This outfit consisted of an old wagon and a new cart with their teams and six animals for riding and driving purposes. The outfit came from Colonia Lerdo, Mex., an old settlement on the lower Colorado River. The men were recruited partly from Yuma, partly from Somerton, Ariz., and the rest, who owned the wagon and animals, came from Colonia Lerdo. The Colonia Lerdo men were hired for the reason that it was thought good policy to engage some men well acquainted with the desert. It soon developed, however, that the wagon, animals and men were unfit to carry out the work satisfactorily and the whole party had to be reorganized.

The animals were half-starved and not fit to draw a heavy wagon load through the deep sand. The men, especially the Mexicans from Colonia Lerdo, proved an entire failure. They were accustomed to the free life of a cowboy and hunter and could ill adapt themselves to the strict discipline of a surveying camp.

As typical of the experiences a surveyor in this part of the world is apt to meet under similar circumstances, I will mention one of this party in particular, who had served as a guide for hunting parties and at first consideration would have appeared to be a valuable accession to the party; but he thought everything should be done according to his ideas, and he was an extremely lazy fellow and a drunkard besides. He tried to scare my by saying that without him, who knew every waterhole in the desert and every place to dig for water, we would soon suffer from thirst and be forced to abandon the survey.

Nevertheless, I did not hesitate to discharge the whole outfit with the exception of one man, a cowboy from Lower California, who knew some of the waterholes. He proved a willing worker and stayed with me through the whole survey. The reorganized party consisted of 9 men, 11 mules, 1 horse and 2 dogs, as shown in the photograph, Fig. 1.

It was not until Dec. 12 that I finally left Yuma, heading southerly across the desert toward Monument 204 of the international boundary between the United States and Mexico, which was to be the starting point of a survey based on the following instructions from the Mexican Government:

The survey of the land situated in the District of Altar, State of Sonora, to be executed as follows:

(1) Point of departure is Point I, which is the intersection of the astronomical meridian through Monument 204 of the international boundary line between the United States and Mexico and a parallel line to said boundary line at a distance of one kilometer.

(2) Thence from this Corner I along the astronomical meridian until the rim of the mesa is reached, to a point II; it is understood that this line must not interfere with any lands now privately owned, which must be proven in the report to the government by locating the nearest corners of the said land in regard to the west line of this survey.

(3) Following the rim of the mesa in a southeasterly direction to Corner III, determined by the condition that the astronomical meridian III-IV intersects the line I-IV in a point IV in such a way that the perimeter I-II-III-IV-I includes an area of 118,737 ha. 27a. 26 centia, with a toleration of 3 per mil.

The report must be accompanied by photographs, original field notes and maps. On the final map the lines I-II and III-IV must be parallel lines. The survey is further subject to the rules and regulations as prescribed for the survey of national lands.

These instructions for linear measurements require that the sides be measured directly, provided that the inclination of the ground is not greater than 3% and of such character that tape measurements can be made without the use of a plumb bob, and that the opening of blazes is unnecessary. Tape as well as stadia measurements are to be repeated at least once. If the foregoing methods are not practicable, triangulation is to replace them. Corners as well as line markers must be constructed as permanent monuments. In the case of triangulation, all triangulation points must be permanently marked. If the lines are measured directly, permanent line monuments must be set from two to four kilometers apart.

*46 Schottenfeldgasse, Vienna VII, Austria.

In the case of triangulation no angles smaller than 20° or larger than 120° are to be used. The base lines are to be measured with a steel tape, provided with a tension measure and applying the correction for temperature. The precision of the length measurements must not be inferior to those allowed for stadia measurements. Angles are to be measured to the nearest minute, following the practice as indicated on the maps of the adjoining lands.

The boundary lines of national land grants are always required to be referred to some point of known geographical position. In case this is impossible the geographical position of one of the corners must be established.

Description of the Survey

Survey work was commenced Dec. 14, 1911, at Monument 204, international boundary, about 20 miles south of Yuma, Ariz. The instruments used were a Gurley's surveyor's transit reading to 1′, with a complete vertical circle also reading to 1′; a Gurley reconnoissance transit, with one vernier on the horizontal limb, reading to 1′ and with a reversion level and diagonal prism and sun glass, and a vertical circle also reading to 1′; two 50-m. steel tapes; one spring balance; one thermometer; one aneroid barometer reading to 16,000 ft., two sets of surveyors' pins; one stadia rod; range poles; one metallic tape; one set of drawing instruments; an Elgin watch, 23 jewels; and several reference books, a dozen blank field-books; drawing paper; a celluloid 8-in. protractor, and other necessary drafting utensils. The only fault to be found with the surveying instruments was the unprotected vertical circles, the graduations of which suffered from the prevailing climatic conditions.

The first field operation consisted in the careful adjustment of all the instruments. The next operation was the establishment for a reference line of the true bearing of the international boundary from Monument 204 to Monument 203. For this purpose two sets of sun observations were taken, each consisting of six observations, one set with telescope direct and the other with telescope inverted. The geographical position of Monument 204 was scaled from a plan of the international boundary line, as I did not have at that time the data on its position, which were later obtained from the U. S. Government.

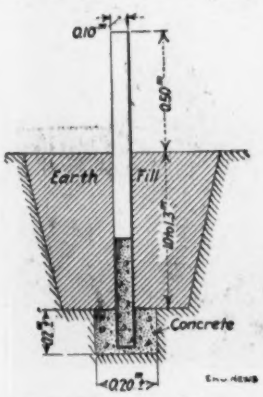

Fig. 2. Type of Monument Used

Corner I of the tract to be surveyed could now be established. The calculation of the distance on the astronomical meridian from Monument 204 to Corner I made the distance 1061.05 m. This distance was measured three times on an offset line with a 50-in. steel tape. The ground was level and cleared of any bushes.

Monument—The permanent monuments consisted of old boiler pipes, 4 in. in diameter, cut into lengths of about 6 ft. Galvanized-iron pipes 1½ in. in diameter with a cross-piece on the bottom would have been far superior to the heavy, rusty boiler pipes, but the promoters furnished the cheaper boiler pipes notwithstanding that the added cost of transportation and distribution made them the most expensive in the end.

Four wood forms and about 50 sacks of cement were included in the outfit with the intention of inclosing the pipes in concrete monuments. Corner I was thus constructed, but owing to the difficulties of transportation the forms and most of the cement were abandoned near the first corner, and they were afterward taken back to Yuma.

All monuments after the first were constructed as shown in Fig. 2. In this manner about six monuments were built with one sack of cement. All the pipes were given two coats of black paint before being set, and the station numbers were painted on in white. The monuments were planted by the survey party as the work progressed.

The West Line—The extension of the astronomical meridian to the south is 53,786.6 m. This was measured partly with a 50-m. steel tape and partly by stadia. Four to five men were used for actual field work. When giving the line one man stood behind the instrument to repeat my hand signal with a large flag to the flagman ahead, who had a similar flag.

The head flagman had already been given orders to walk in an indicated direction or to line himself roughly by other line signals, so that he was always near line. He was provided with a good fieldglass and went ahead as far as he could and still be able to distinguish the transitman's signals. This was often difficult because of the vibration of the air due to the burning sun. The flagman was accompanied by one or two axmen who cleared the line of bushes for stadia sights and opened blazes when the distances were measured by tape. The signalman behind the instrument was also provided with a fieldglass, so that he was able to watch the movements of the head flagman.

The line was thus staked out, always reading the approximate distance between hubs by means of a pedometer and a speedometer on the wheel of the cart, which read to tenths of a mile. The head flagman was always told which line point would be a permanent monument and was directed to choose a prominent point, so that the monument could be seen for a great distance in either direction along the line. Every instrument station was marked with a flag signal. As soon as two permanent monuments were established and their direction checked, these intermediate signals were recovered (for economical reasons).

All the permanent line monuments were marked by large signals carefully set to line by instrument. These signals were 22-ft. poles, made by nailing together two 12-ft. poles. The flag at the top was made of 1x3-in. boards, 3 ft. long, on which were fastened alternate strips of red and white cloth. Fig. 3 shows one of these signals behind Corner IV. Such signals could be seen early in the morning and late in the afternoon for a distance of from 6 to 10 km.

The tape measurements proved much more satisfactory than the stadia readings. Sometimes two stadia measurements gave such differences that a third measurement had to be made. The accuracy of the stadia measurements depended largely on the time of day at which they were made. Early in the morning a reading could be accurately made at 200 m., but later in the day good results could not be obtained for distances greater than 130 m. About the best average result obtained with stadia was an accuracy of 1 in 1000, while tape measurements averaged about 1 in 3000.

Sun observations were taken from all the permanent monument stations, taking back- and foresights to the monuments on either side, thus the line between the three monuments was constantly checked as to its straightness. At the same time all signals at intermediate instrument positions were controlled and any deviation from a straight line was immediately discovered. Besides sun observations occasional observations on Polaris at or near

FIG. 3. MONUMENT WITH LINE SIGNAL ERECTED, CORNER IV, NORTH LINE

elongation were taken; in case the observation was not taken at elongation the star δ Cassiopeia was also observed and the azimuth computed by the method given in Hosmer's "Azimuth." Sun observations, however, have the great advantage that they may be taken along the line as the work progresses, while star observations always involve many inconveniences.

To fulfill the condition of the government instructions, the positions of those corners of the neighboring lands which came nearest the west line were computed. The coördinates of four of these corners were then computed in regard to the nearest line monuments of our survey as zero points. The coördinates were then measured on the ground and a search made for the corners. All four of these corners were found although there were considerable discrepancies between the calculated distances and the measured ones, in some instances being as much as 30 m. These corners were marked with cast-iron monuments as shown in Fig. 4. Some of these monuments, however, were not found set in their intended positions.

SOUTHEAST LINE—The southeast boundary of the tract follows the rim of the mesa and is consequently a broken line. Similar methods of surveying were employed as on the west line. The angles, as a rule, were read to even degrees. Before establishing the permannt monuments, the angles were checked by repetition, one set with telescope direct and the other with telescope inverted. Before definitely establishing the southeast Corner III calculations of the area were made from given data on the north, west and southeast lines with an assumed southeast corner.

THE EAST LINE—Only two permanent monuments were established on the east line, at +3239.2 m. and +7764.1 m. distant from Corner III. The last point on the line +8544.4 m. was marked with a high flag. This line was measured partly by tape and partly by stadia. Sun observations were taken as before to keep the true meridian.

The survey of this line was abandoned at +8544.4 as the line beyond was crossed by immense sand dunes and the land would have been worthless to the promoters. I proposed to extend the west line farther to the west to meet the corners of the adjoining land and extend it farther south so as to narrow the tract and exclude the sand dunes and still have the necessary acreage, but this could not be done without the approval of the government. I was preparing to continue the east line through the sand dunes when I received orders from the promoters to abandon this work and return to the north line and set the northeast corner by calculation.

This was contrary to the instructions from the government for it would be impossible for me to show the error of closure. Nevertheless, I was compelled to comply and return across the desert to Monument 204 of the international boundary.

THE NORTH LINE—A part of the north line had been measured by me while camping near Monument 204. The distances between the monuments on the international boundary I was compelled to get by scaling a government

FIG. 4. CAST-IRON MONUMENT CORNER OF ADJACENT SURVEY, NEAR WEST LINE

map. The north line was measured throughout with a steel tape. The procedure was as follows:

From a point about 1 km. from Monument 204 on the line between Monuments 204 and 203, a right-angle offset of 1000 m. was made and a signal posted. This line was prolonged until a point nearly opposite Monument 203 was reached, when sights were taken to the center of the monument and the angle between the line prolonged and the monument measured (Fig. 7). The re-

maining distance to the point opposite the monument was calculated by the relation: $x = 1000\ cos\ (180° - d)$. Our line monument M_{203} was thus provisionally established. Then the distance on the ground to Monument 203 was measured and the mean of two 1000-m. measurements adopted for the final location of our line monument M_{203}. The line stakes in the rear were then offset to the true line in proportion to their distances from M_{203}. This, of course, did not affect appreciably the measured distances along the north line. Monuments M_{203}, M_{202}, M_{201} were thus established. My measured distance along the north line from points opposite international boundary Monuments 204 to 200 was were calculated, based on the previously made measurements. The sides of the tract were computed, allowing for the convergence of the meridian, and Corner IV set so as to include an area of 118,580.7156 ha., which was inside the toleration.

Character of the Surveyed Tract

The character of the mesa of the Sonora desert changes much the farther south one goes. Near the boundary line the land is practically level for a width of about 14 km., then sand dunes begin to appear. The land bought under this survey it was intended to irrigate by pumping water from the Colorado River. The elevation of the

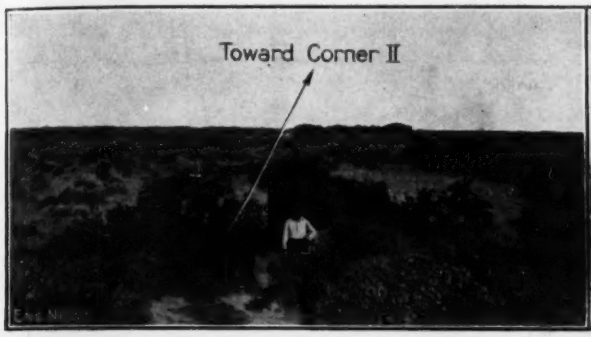

Fig. 5. Southeast Line, Looking toward Corner II of West Line

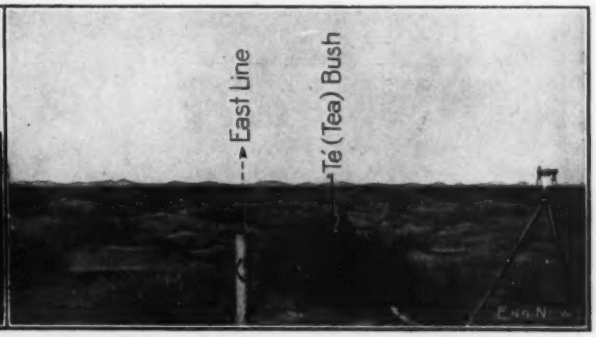

Fig. 6. Monument on East Line, Looking North, Sand Dunes in Distance

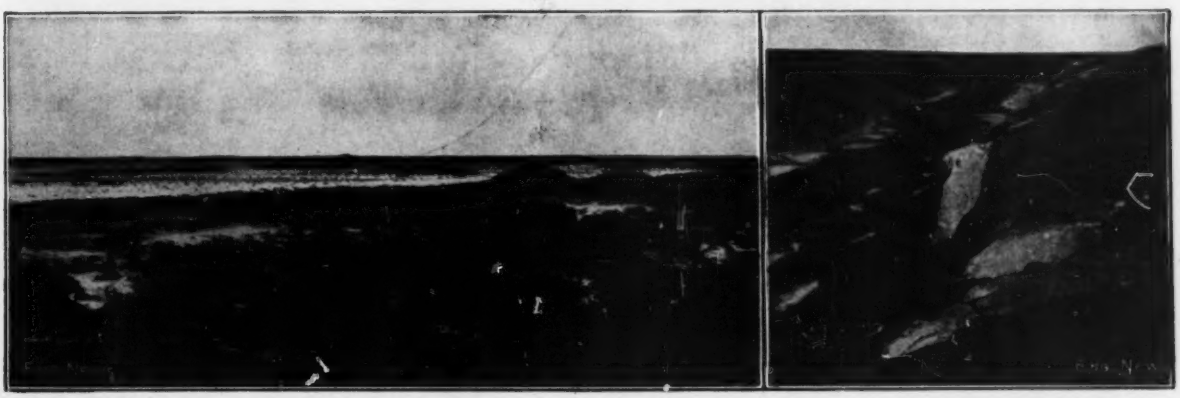

Fig. 8. Views of the Western Rim of the Mesa and the Salt Beach Extending to the Gulf of California

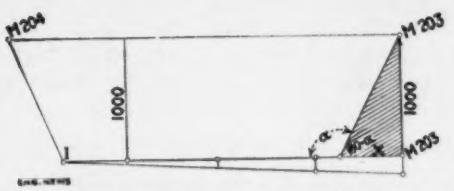

Fig. 7. Showing Method of Setting Monuments on North Line

20011.1 m. compared with 20002.4, the astronomical distance afterward obtained from the report of the Boundary Commission.

To establish the northeast corner of the land by calculation as directed, the convergence of the meridians through Corners I and III had to be established. For this purpose the latitude and longitudes of Corners I and III

land near international boundary Monument 204 is 131 ft. above sea level, and about 90 ft. above the Colorado River. Near Corner IV the elevation is about 197 ft. above sea level and 156 ft. above the Colorado River. The mesa falls off steadily to the south and west and near Corner II the rim of the mesa is formed by many sand dunes. The land itself slopes greatly toward the salt marshes of the "Playa Salada," as shown in Fig. 8.

The "Playa Salada" or Salt Beach forms along its borders with the mesa extensive swamps, some of which were originally included in the land grant. During gold fever days in California, Puerta Isabel, to the southwest of this tract, was a busy little harbor and had direct steamer connections with Yuma, Ariz., but today there is no port and no indications of past activity save a few old hulks of vessels half buried in the ground. The Playa Salada is practically level and its elevation above sea

level and the water of the Colorado River is only a few feet. The presence of driftwood near the foot of the bluffs on the rim of the mesa led me to believe all this area was submerged at certain stages of the tide, and that the sea was gradually receding.

Corner III was located near a dry cañon, Fig. 9, and in this part of the desert we found many petrified trunks of trees, some 6 to 8 in. in diameter, which indicated that the land had not always been barren. Beyond the last monument set on our east line, the line was crossed by immense chains of sand dunes separated by strips of the receding waters. The mesa formed at one time the coastal plains along the Gulf.

In our survey on the mesa we often met with that peculiar phenonenon known in almost all deserts under the name of mirage. It was observed especially well in the early hours of the morning. Just before sunrise the distant mountains of Arizona often assumed queer shapes. One could see castles with high towers, sometimes whole cities of them. Soon after sunrise the castles and towers began to shrink and flatten out and the mountains assumed their normal shapes. On a few mornings a particularly beautiful mirage was seen in the direction of Lower California, which had the appearance of a lovely river, bordered with tall trees, and even the reflection of the trees in the water was easily distinguished. Sunrise and sunset in the desert are indescribably beautiful and one never tires of the changing colors of the mountains and of the sky.

FIG. 9. DRY CANYON NEAR CORNER III

FIG. 11. CORNER III LOOKING NORTH

FIG. 10. SAND DUNE SECTION OF THE SOUTHEAST

plain (Fig. 9). To the south and east the land as far as could be seen was covered with nothing but these huge sand dunes. To the north the chains of sand dunes grew smaller and smaller, forming the transition to the plains.

These great sand dunes have smooth surfaces and in the distance show smooth curved lines (Fig. 10). The sand is in constant motion. One's footprints are quickly lost. The bright sunlight on the white sand is dazzling, and this, coupled with the wonderful forms the heaps of sand assume, makes one dizzy to look around. This whole region in former times was undoubtedly covered by the waters of the Gulf of Mexico, and the different chains of sand dunes probably mark the successive coast lines of

The soil of the mesa is nearly uniform and consists partly of what is known as Yuma sand and of Fresno gravelly sand. The subsoil of the mesa, as evidenced in the dry cañon near Corner IV and on the higher bluffs along the southeast line, is of the same sandy nature to a great depth.

CLIMATE, VEGETATION AND ANIMAL LIFE

The time of year for the survey could not have been better chosen. During the month of December and the first part of January the temperature fell as low as 17° F. on several nights. It was an exceptionally cold winter for the Yuma desert, which has the reputation of being the hottest place in the United States. It is always warmer on the mesa than in the Colorado River Valley. The nights, however, were indeed so cold that I had to send for a camp stove to keep warm enough to do my office work. The days were, with very few exceptions, delightful; during December we had a few sand storms,

which for a few hours at a time made work impossible. Several times it tried hard to rain but without success. From the latter part of January on the nights were much milder and while working and camping along the

FIG. 12. CAMP ON NORTH LINE SHOWING GROWTH OF GREASEWOOD AND "GERONDILLO"

salt marshes the mosquitoes were such a pest we had to keep smudge fires burning each night.

On the mesa the only vegetation which is able to survive the climatic conditions are greasewood, a plant called "gerondillo" and another called "té" (tea) by the Mexicans. Near the international boundary line Monument 201, a tall grass called "galleta" was found, of which our animals were quite fond, and which could possibly be made to replace alfalfa as a fodder. It is said to have been highly thought of at the old U. S. Cavalry posts which formerly existed along the border. I think this part of the mesa must have some underground feeders from the mysterious "Black Lake" near boundary Monument 199. One can easily imagine what could be grown on these lands if properly irrigated. This same "galleta" grass was found in places along the west and south lines (Fig. 3). Rain is almost unknown in this part of the Sonora desert.

The "té" is a leafless brush growing in clusters. It was found in spots so barren that even greasewood could not grow, and it even tried to establish a foothold upon the sand dunes. The Mexicans boil this plant and drink the extract, claiming that it purifies the blood. We all drank some, but the most I can say for it is that it caused no ill effects.

Parts of the line were covered with heavy growths of "mesquite." The best way to get a line through these "mesquite" thickets, upon the advice of my Mexican axmen, was to set fire to them the night before. In the morning the survey could be continued without difficulty.

In the same part of the desert was the only species of cactus which we found, called "challa." This cactus is quite small, of very grotesque shape, and covered with beautiful blossoms and long needle-like spines, which caused very painful wounds. Our dog "Shep" suffered the worst from these for he jumped right into a cluster while chasing a coyote.

A part of the tract in the vicinity of one of our line monuments had the aspect of a pasture rather than a desert, with several grasses growing about with "mesquite" and "bean trees" thrown in. The fruit of these trees is a nourishing food for animals and I noticed that it was collected on farms in the Colorado River Valley for feeding the stock. Away from the salt marshes on the west, however, vegetation soon begins to get scarce. The farther south one goes from the international boundary, the more rare vegetation becomes. The great salt plains along the water front are covered with tall reed grass and other plants only along the edges next the mesa, and are for the most part devoid of vegetation and of a reddish brown color.

We had no startling adventures with wild beasts and with the exception of two giant tarantulas, a very venomous spider, whose hairy body is a dark green color, resembling a crab in general appearance, no poisonous insects or serpents were encountered. Natives fear the bite of these tarantulas much more than that of a rattlesnake.

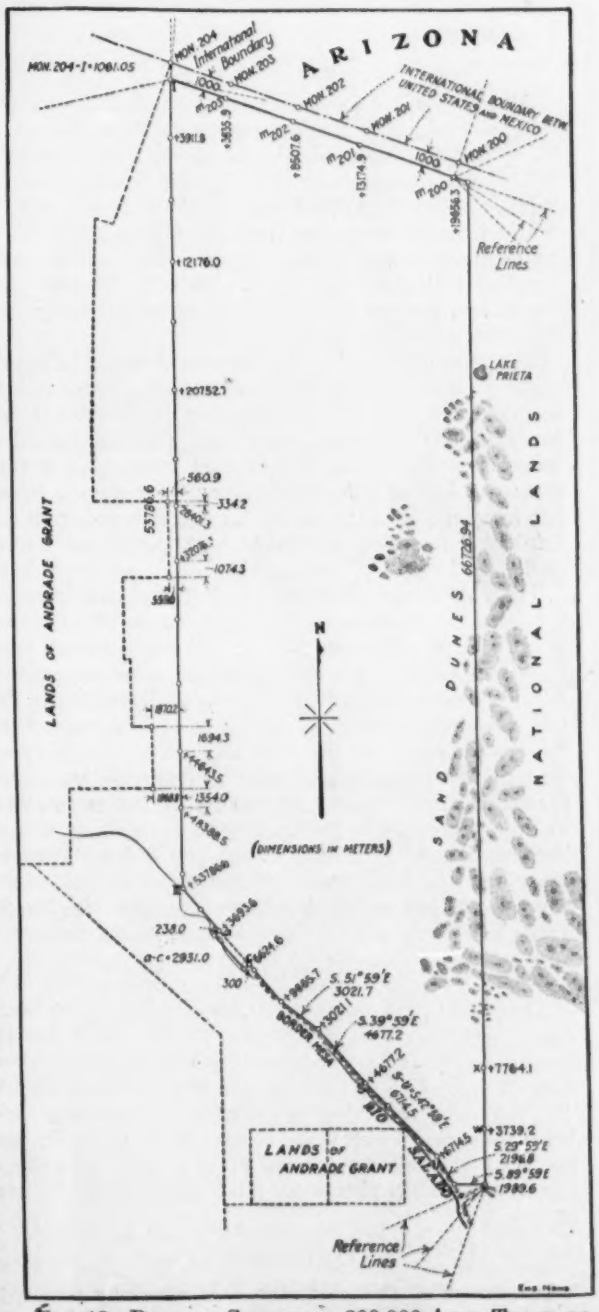

FIG. 13. PLAN OF SURVEY OF 300,000-ACRE TRACT IN THE SONORA DESERT

Coyotes and foxes were quite common and paid frequent visits to our camp. The coyotes were very cunning, and after some had succeeded in drawing off our two dogs, others would invariably be found investigating our commissary department. We also saw small herds of wild donkeys, and we captured and subdued a wild horse in the course of survey of the southeast line.

The salt marshes of the Playa Salada are inhabited by countless thousands of water fowl. Wild geese were sometimes so numerous along the edges of the mesa that at a distance they gave the appearance of a great white sheet spread out over the land. However, they always had outposts stationed, and we found it very difficult to get near enough to kill any, which was tantalizing to hungry men.

The Food and Water-Supply Problem

All supplies had to come from Yuma, Ariz., and everything was subject to Mexican custom duties, and so everything had to be examined and checked at Monument 204 of the international boundary by the Mexican officials. This complicated what would still have been a considerable problem, for the most serious difficulty of an engineer in this class of work is to keep his party supplied with food and water. To keep the men contented and prevent them from deserting is another and somewhat dependent problem.

The supplies were brought by special teams to supply stations established along the west line. Water had to be found and brought from considerable distances, sometimes from the nearest ranch and sometimes from a waterhole. Water was easily found by digging two or three feet in the valley lands along the Colorado River, but in most places the water was brackish and unfit for drinking. A spring was found near one of our monuments.

The approximate location of all camps was planned beforehand. As soon as one camp was established the location of the next one was decided upon and all supplies not required for immediate use were sent ahead. About a dozen 50-gal. barrels were distributed along the line ahead at prospective camps and these were kept filled with water for the men and stock. In this way we were always assured of water at each new camp and the animals could be used for moving the equipment. The water was brought in twenty 10-gal. barrels by a wagon kept continually in service. Meat was had at Colorado River ranches during most of the survey. The whole party, men and animals, returned in good health to Yuma in February.

Cost Data

The survey had occupied 65 days, with 57 working days. We had surveyed 246.4 km. of line (with double measurements) or at the rate of 4.32 km. per working day. The cost of the survey was approximately $6000 (U. S. currency), including everything. The area surveyed comprised 118,580 ha., so the cost in round figures was approximately 5c. per ha. (2.47 acres). The Mexican Government's charge for land surveys is 12.5c. per hectare.

Conclusions

In surveys of this character it is especially essential to have a very careful reconnoissance, the need of which in this instance was severely felt. The short time allowed to complete the work made it impossible to arrange beforehand many things, which done during the survey add much to the cost and inconvenience.

An engineer embarking on the survey of a large tract in the desert should have two men with him in whom he can place absolute confidence. The one is the camp superintendent and the other a surveying assistant, who is able to carry out part of the technical work while the chief of party is attending to other duties.

The writer was fortunate in having a first-class camp superintendent, an American plainsman from Yuma, Ariz., who had lived a number of years in the Southwest and in Sonora, who was fully acquainted with conditions in the desert and proved capable at different times of solving many arising difficulties. The writer, however, did not have an assistant engineer and this need was often felt.

Good-Roads Construction in the Outlying Parts of Chicago

The outlying districts of the city of Chicago have many lines of streets which form continuations of country roads, and which are practically unimproved where they extend through the more distant and thinly populated sections. At certain times of the year some of these connecting roads are in such condition that the suburbs which they reach are cut off from ordinary vehicle traffic, to say nothing of the great and increasing automobile traffic.

The traffic with these suburbs has increased to such an extent that the demand for good roads leading to the paved streets had to be considered by the city authorities, and (in relation to the movement for good roads throughout the state) a plan was adopted for the improvement of connecting links between the city's paved streets and the main country roads extending beyond the city limits. This work is under the direction of the Bureau of Streets of the city's Department of Public Works, and for information concerning it we are indebted to Walter G. Leininger, Superintendent of Streets.

In view of the mileage and the limitations of expenditure the only paving that could be considered was one that would be cheap, could be constructed rapidly, and would require no great amount of preliminary work. For some years the Bureau of Streets has carried on macadam-road construction, so that it had a suitable organization in working order, and this was adapted to the new road-improvement work.

Each gang consists of approximately 18 men, with two foremen, and equipped with 15 to 20 teams, a 12-ton steam or gasoline roller, a sprinkling cart and a road grader. About 14 men are kept on the railway cars, unloading stone into the dump wagons. The others prepare the subgrade, spread the stone and do the other work of construction. The length of haul from cars to the road varies from ½ to ¾ mile. The dump wagons are of 3-yd. capacity, but as the roads over which they travel are in many cases in very bad condition it is generally impracticable to carry more than 2 yd.

Such a gang will construct about 1000 sq.yd. of road per day, or the equivalent of 500 ft. of 18-ft. roadway. There are three regular gangs operating directly under

the Bureau of Streets, and these take care of the larger jobs. There are also three smaller gangs under the ward superintendent, and these take care of the smaller jobs and repair work. The same organization is employed for maintenance, and as this latter work can be carried on only in the spring and summer the same gangs form the maintenance force as well as the construction force.

The first part of the construction work consists in preparing the subgrade, which is mainly soft earth or clay. This work is done sometimes by hand, but generally by a road grader, which is set to cut to a depth of 4 or 5 in. at the edge. All holes and ruts are filled in this way, and the subgrade is then well rolled.

Upon this subgrade is laid a 6-in. bed of large stone, averaging 4 to 6 in. in size. After being rolled, this is covered with 3 to 4 in. of stone averaging 1½ to 2 in. diameter. This is spread with limestone screenings, and then watered and rolled to insure thorough filling of the voids. Then the surface is again covered with screenings and rolled. Care is taken to roll it while wet, and the rolling is continued until the water flushes in front of the roller. When this occurs, the surface is swept to insure filling all minor voids and the road is barricaded against traffic for four or five days. At the end of that time the roller is again used to insure a smooth and firm surface.

DOUBLE-HANDLED TWO-MAN RAKE FOR SPREADING AND LEVELING STONE ON MACADAM ROAD CONSTRUCTION

The specifications call for dry, clean, hard limestone, free from discolorations, clay, dust or foreign substances. Quarry strippings are not accepted. The stone is spread as far as possible by opening the end of the wagon box 6 or 7 in., and allowing the stone to flow out as the wagon moves along. It is finally spread and leveled by two men using the double-handled rake shown in the accompanying cut. This is more convenient than the use of two ordinary rakes crossed or locked so as to work as one tool.

Where the subsoil is bad and cannot be rolled, the bed of large stone is laid and covered with the coarse screenings. This is then rolled and the street is left to settle. The following year this foundation will be good, and the upper course of stone can be laid, rolled and finished as described. The finished road has a paved width of 18 ft., with a crown of about 6 in. and a thickness of 10 in. at the middle and 6 in. at the sides. At unimproved cross roads, the edges of the stone paving are protected by broken stone aprons 10 to 12 in. wide, about 4 in. thick and tapering to the outer edge.

The line and grade are made to conform to the old road, in order to save expense. For good surface drainage, the berm is made level with the road and is given a slope of about ½ in. per ft. to the side ditches, which are 3 to 4 ft. deep below the crown of the road. This depth is sufficient for drainage of the subsoil.

In 1912, 83,236 sq.yd. of macadam road was built, at an average cost of 56c. per yd. At the end of 1912, a total of 55 miles had been completed. Although water-bound macadam seems to be the only cheap pavement for outlying roads, it is not advised for city streets as the dust nuisance is a great source of complaint. The only cheap repair that can be made is to oil the surface and cover it with sand, ¼-in. limestone screenings or granite chips. Heavy oil is more satisfactory where great care is taken to apply it hot and to cover it immediately with screenings. The complaint which sometimes follows is due to the fact that heavy oil penetrates slowly and the coating on the surface is tracked by pedestrians into houses and upon sidewalks. In 1912, 5,311,144 sq.yd. of macadam pavement were oiled at a cost of 1.35c. per sq.yd. In all, 1,353,753 gal. of oil were used.

The oil used for macadam roads is required to meet the following specifications:

Specific gravity, 20° to 22° Baumé at 77° F., unless the specific viscosity is 1.50 to 1.85 at 212° F. when tested on the Eugler viscosimeter.

Fixed carbon not more than 8%.

Flash point, not below 275° F.

Loss at 212° F.; none.

Mineral matter or ash; none.

Soluble in cold carbon bisulphide, not less than 99%.

Loss when heated five hours at 325° F. not more than 5%. Residue must flow at 77° F. and must show no coarse crystals.

Residue from heating oil for five hours at 400° F. must flow at 77° F.

After heating oil at 500° F. to a penetration of 100 at 77° F. there must be left at least 25% of material which shall have a ductility of more than 10 c.m. at 77° F.

Repairs can be made in the following manner: Where the road is of fair thickness, it can be scarified and covered with additional stone, the construction being the same as on new work. The scarifying may be done by a device attached to the road roller. Where it is advisable to lay 3 in. or more of stone no scarifying is found necessary. Where the pavement is in fair condition except for an occasional hole or rut it can be repaired by cutting out the hole, placing stone and covering this with asphaltic cement or tar. About 1½ gal. per sq.yd. is found to be efficient.

For maintenance (as distinguished from repairs) the best method is to place the roads under the patrol system. By this system one man has to take care of the maintenance of roads in a certain district. Care must be exercised in cutting out holes, for without proper edges the repair will not last. Merely filling a rut or hole does not insure a permanent repair.

This patrol system has not yet been put into effect, but this will be done very soon. Each man will maintain about three miles of road, with supply stations located at ½-mile intervals. These stations will be equipped with necessary boxes for the material. Each patrolman will be furnished with a ½-yd. hand cart for the stone, a tamper, broom, shovel, pick, tar bucket, water bucket and wheelbarrow.

Fatalities in Coal Mines during 1912 numbered 2360 as against 2719 for 1911, according to the Bureau of Mines, Washington, D. C. The fatality rate was lowered from 3.73 in 1911, to 3.15 per thousand men employed in 1912. The fatality rate in a number of foreign countries during a period of ten years (1901 to 1910 inclusive) are as follows: Great Britain, 1.36; Germany, 2.11; France, 1.69; Belgium, 1.02; Japan, 2.92. The rate in the United States during the same period was 3.74.

Technical Education and Professional Ideals*

By W. F. M. Goss†

The American technical school stands today on the threshold of great achievements. Captains of industry no longer look in doubt upon the technical graduate; they give him work. Public confidence in the new education has been won and its permanent support is assured.

An attempt to outline the problems of the technical school must recognize the fact that great advances in any art or profession await the coming of great men. The modern need in the field of engineering is for men who can perform the exceptional task; for men who are safe keepers of their brothers' interests; for men whose qualities of character are so sound and strong that they instinctively perceive the way of truth and follow it; for men whose activity and understanding detect the defects in established practice and find a way to improve that practice; for men who so well understand the fundamental principles of science as to be able to predict the effect of the next step before it has been taken; for men, in fine, who can rank with the world's great leaders in science and industry. I conceive it to be the prime purpose of the technical school to make its contribution toward the development of such men.

It would seem unnecessary to remind an audience of professional engineers that the achievements of the technical school center in the quality and strength of its instructional staff. The present-day need is for increased strength of staff. All honor is due to the fine coterie of men who, undertaking the work of the technical school a quarter of a century or more ago, have adhered to their task in the face of discouragement at great personal sacrifice and with a spirit that has been fine and even heroic. But retrospect, however satisfactory in itself, will not solve our future problems. Great leaders in the work of the technical school are few, and the unfortunate fact is that throughout the years that are past, few men have been in training where the service now demands many.

There are many reasons for this. An important one is to be found in the limitations which have been put upon the salary budget. The man who builds great bridges or who directs the activities of a great industry, is none too gifted to guide the efforts of ambitious youth, and hold before the student large ideals of life. And yet, in the industries a man may receive in a month an amount equivalent to the annual salary of many college professors.

It is clear to all who know the field that while the school must suffer some years to come, through its deficiencies in the past, a new policy should be entered upon as speedily as possible. Few among American technical schools can today offer professorships which in themselves are sufficiently attractive to justify a young man in securing for himself so elaborate a preparation. The remedy cannot ordinarily be applied by the college itself; it must come as the result of interest shown at the sources of financial supply. In the great state universities it must be the result of public appreciation of the need; you, I know, will realize that public appreciation, though a plant of slow growth, may be cultivated by many different agencies.

Not only is it required in the interest of higher efficiency, that the American technical school have an instructional staff of the highest possible quality, but it is also required that such a staff be not overburdened with routine duties. If the engineering graduate sometimes degenerates into an animated slide rule, may it not be possible that he has been instinctively led to such a career through ill-conceived treadmill processes in the classroom? If so, the remedy is to be found in reforming the work of the classroom, and one sure road to such a reform is that which opens the way whereby men of large caliber may have time in which to impress themselves upon their students.

The technical school is greatly in need of simple living and high thinking. The simple living is easily attained unto, but high thinking, in its essence, involves a certain element of leisure, or perhaps freedom in the choice of one's occupation, which, after all, is but another term for leisure, and no atmosphere surcharged with high thinking can prevail on a campus where every individual student and professor is perpetually keyed up to concert pitch in an endeavor to accomplish an assigned task.

It is altogether possible that in our present-day routine too much time is given to things which are simple. Much that is now studied may perhaps be read. The habit of studying intensely a few books to the entire exclusion of the great mass of historical and biographical engineering literature affords the student but little opportunity of acquiring a habit of rapid and intelligent reading, which in itself is an accomplishment worth striving for. The practice of the shop, laboratory, the drawing-room, the surveying field and the study of descriptive texts, rightly interpreted, are important adjuncts in the training of the engineer, but the time has ceased to be when such activities constitute the chief characteristics of the technical school. The years in college must be spent in acquiring an understanding of principles and in the development of those aspects of theory which are difficult to acquire after one's college days are over.

The technical school cannot content itself with routine service. It must enshroud its classroom and its laboratories with an atmosphere of scientific achievement; its professors should be leaders not only as classroom instructors, but nation-wide leaders, even world-wide leaders, in the complex and highly diversified fields of the science which they represent.

A laboratory which has been evolved through the activities and desires of a master is not only priceless for the school that possesses it, but necessary as a source of information of the highest value, to the field of practice which it is designed to serve. The chief engineer of a great railway system, in writing of the work of a certain college laboratory, has recently certified, in terms which are clear and emphatic, to the value of principles affecting the design of certain structures which had been developed by the laboratory and extensively used by the corporation which he served. Assuming that the country has need of research laboratories for purposes quite apart from the work of instruction, it can, I think, be shown that the technical school constitutes the most promising agency in our national economy upon which to place the responsibilities incident to their creation and maintenance.

Educational institutions of high standing are sending out each year into the engineering pursuits of our country more than 2500 graduates, nearly 1000 of whom enter the field of mechanical engineering. The technical school is, in fact, recruiting and otherwise stimulating the engineering work of the country.

The work of the school tends to emphasize the dignity of the calling. Professionalism as distinguished from the art of practice of engineering is after all a question of quality. Professionalism is breadth! In our ambition to have a large part in the world's affairs we somestimes forget the fundamental source of power. We sometimes think too much of honors and too little of service; we desire the position before we are qualified to discharge the responsibilities it imposes. Our strivings must be for fitness. Progress in advancing the ideals of our profession will depend upon the character of the service rendered by the members of the profession. If every practitioner in the field of mechanical engineering possessed character and ability responding to our highest ideals, no one would question the right of mechanical engineers to regard themselves members of a profession.

A second service, which the technical school is rendering the profession of engineering is that of contributing to the sum of its scientific data. Engineering as a science has made progress by leaps and bounds; rule of the thumb has given way to the rule of the mathematician and investigator. But notwithstanding all that has been accomplished in this direction, there are as yet but few departments in the field of mechanical engineering in which the basis of design is actually perfected.

We, as engineers, need to train ourselves to a condition of mind which will make studious processes less difficult than at present. Along this line of endeavor the researches of the technical school will aid the profession by establishing new standards of proficiency, and by making new contributions to the existing fabric of facts.

Whatever may be the nature of the contribution of the technical school to the ideals of the profession, the extent of the contribution is a function of the quality of the school. Better instruction in the school must supply better recruits for the profession, greater activities in scientific researches on the part of the school must operate to increase the facility with which the problems of the engineer are solved, and influences in the school which tend to extend the student's horizon and broaden his sympathies, will in due time make their impress upon the professional life outside of the school. That is, efficiency in technical education is a factor in the development of professional ideals, and hence a matter of prime importance to this society, as well as to all other organizations of engineers.

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*Condensed from the Presidential address before the American Society of Mechanical Engineers.

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An Earth Slide at Bellevue, Penn. and Suggestions for Arresting Its Further Progress

By R. P. Forsberg*

The slide described herein is located in the residence section of Bellevue, Allegheny Co., Penn., and has caused great inconvenience and expense to the residents in the affected district as well as concern to the municipal authorities. The writer was called in to report upon the conditions and to recommend methods for arresting the further progress of the slide. The accompanying description is based upon the report which he presented to the municipal authorities.

Slide Conditions

The area or zone embraced by the slide is indicated on the map (Fig. 1) and the conditions existing within this zone are as follows:

The vitrified brick paving in the west side of the driveway of Rodgers Ave. for a length of 200 ft., is bulged or raised to a height of 18 in. above the general elevation of the street, rendering this portion of the driveway wholly unfit for vehicle traffic.

The cement or granolithic slabs composing the west sidewalk, have been raised from their foundation and in some cases moved over the driveway curb.

The retaining walls at the foot of the terraces have been overturned and thrown on the sidewalk. The front steps leading to the residences on this side of the street have been moved out of line and toward Rodgers Ave. (see view in Fig. 3). Many of these buildings have been moved out of plumb and their foundations seriously damaged; in one case the house, on Lot 352, has been rendered wholly unfit for occupancy. The surface of the ground extending from the foot of the slope in the rear yards of these residences, to the top of the slope just back of the residences on Summit Ave., is unbroken and there is no evidence of any unusual disturbance. The ditch in the alley between Rodgers Ave. and Summit Ave. is in rather bad condition for properly taking care of the surface water that drains into it from the side hill, and in its present state is rather a menace than a safeguard. At or about the top of the slope there is a break in the surface varying in width from 1 to 6 ft., and extending throughout the length of the zone. This break has been filled in from time to time only to appear again after the spring rains. Rodgers Ave. was graded or opened in 1895, and indications of this slide first appeared in 1908. No movement is evident in the east curb of Rodgers Ave. or the residences on the east side of the street.

The above stated facts represent a rather serious state of affairs for the property owners within the limits of this zone. Serious, not only in the damage that has been wrought, the general extent of which is apparent to even a casual observer, but also in the fact that the residents on the east side of Rodgers Ave., the east side of Summit Ave., and both sides of Dakota Ave. (the next street below Rodgers Ave.), have no assurance that their property, lying within the length of this zone, may not at some future time be similarly affected. The writer does not believe that the buildings on the east side of Summit Ave. will be affected by this slide for many years to come, even though no artificial means be taken to arrest its further progress, for from observation it appears that they are founded on a good shale rock, from which the slide has broken, but it is a moral certainty that under the present condition of affairs the break in the surface of the ground just in the rear of these buildings will continue to appear from time to time, rendering that portion of the property wholly unfit for use and a constant menace to the safety of children playing in that vicinity.

The slopes of Summit Ave. indicate that hard shale rock was encountered in its grading, which fact suggests that blasting was probably employed. If so, it is possible that the resultant shock from such blasting might have been a contributing factor to this trouble, but it is clearly manifest, as will be shown later on, that it was not the cause, and therefore no further attention will be given to it.

Investigation of Geological Conditions

After having made a thorough examination of the exterior conditions it was determined to sink shafts or test holes to determine the character of the material. Two of these test holes were sunk at the locations indicated on the map (Fig. 1) and the depth and character of the material encountered is indicated on the cross section (Fig. 2). The result of these tests was a distinct disappointment to the writer, as he had been led to believe from conversations held with some of the residents of this vicinity that hard rock would be found at a depth of 10 or 12 ft. below the ground surface. With this information as a basis, sketches of a method had been prepared that probably would have satisfactorily taken care of this trouble at a reasonable expenditure, but the result of the findings in the test holes completely overturned these plans.

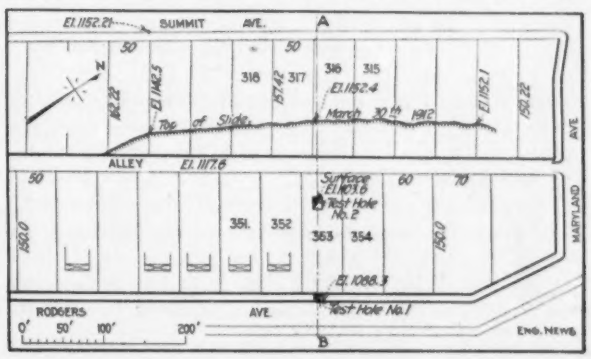

Fig. 1. Map of Portion of Bellevue, Penn., Showing Location of Slide

Test hole No. 1 was excavated to a depth of 7 ft. below the elevation of the sidewalk. At an elevation of 5 ft. below the sidewalk a vein of fire clay about 6 in. in thickness and about the consistency of putty was found. A rod sent down several feet below the bottom of the excavated hole showed hard shale rock to the depth that it was driven. The material above the vein of clay, chiefly loose shale rock, was fairly dry, but at the elevation of the fire clay, water was dripping freely into the test hole.

Test hole No. 2 was started 4 ft. wide and 10 ft. long, at a depth of 9 ft. we reduced this to 4x6 ft. and finished at the bottom with a section 4x4 ft. From almost the

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beginning it was in good shale rock of uniform character, only two small and unimportant seams of clay appearing. The hole was only down 10 or 12 ft. when water first made its appearance and from that time until the test was completed this had to be contended with in large quantities, it being necessary to keep a pump going almost constantly while the excavation was in progress. At a depth of 21 ft. from the surface of the ground a vein of fire clay of the same character as that found in No. 1 test hole was encountered but this vein was 21 in. in thickness. Below this vein for a depth of several feet, as far as a rod could be driven, it appeared to be shale rock of the same general character as that which had been passed. Around the top of the fire clay, water was freely flowing in. Several small veins were discharging from the Rodgers Ave. side of the hole and one large one, about twice the size of a lead pencil, was discharging from the northwest corner. The south side of the hole was practically dry. A test showed that the water was flowing in at a rate of about 6 gal. per min. The water was clear and cold.

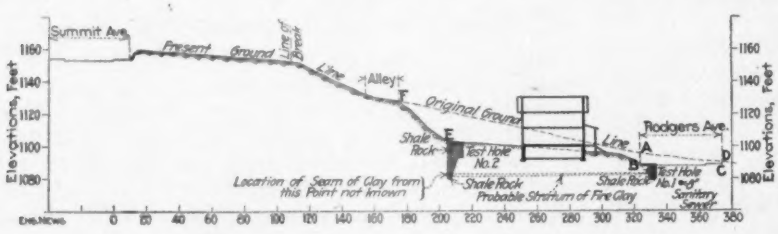

FIG. 2. SECTION THROUGH AREA AFFECTED BY SLIDE
(Section through Lots 353 and 316, Fig. 1.)

To recapitulate—The test hole was driven through 21 ft. of good shale rock in which there was absolutely no trace of a break, the sides of the hole were practically dry for this distance. At 21 ft. from the surface the vein of fire clay was encountered with water freely flowing in on top of it.

CONCLUSIONS AS TO CAUSE OF SLIDE

From these facts the writer arrives at the following conclusions:

(1) That underlying the slide zone, heretofore described, is a vein of fire clay varying in thickness from 6 to 21 in., which vein is thoroughly saturated with water.

(2) That the water comes either from natural veins or from a leaking water main in Summit Ave. Unquestionably some surface water enters the clay, and this should be remedied by an approved system of surface drainage being installed, but the slide would not be arrested by this, as without question the water that is doing the damage is supplied from either one of the two sources heretofore stated, with the probabilities in favor of the former.*

(3) That in connection with the natural conditions existing in this zone the slide, in the last analysis, was precipitated by the excavation of Rodgers Ave. and the plateau above, upon which the residences are built. The excavation of Rodgers Ave. alone would never have started this unfortunate condition of affairs. This statement is made guardedly and only after access was had through the courtesy of Edeburn, Cooper & Co., who were the Borough Engineers at the time that Rodgers Ave. was graded, to the original records, from which were secured data to plot on the cross-section the original ground line. *ABCDA* on the cross-section (Fig. 2) represents the area removed by the grading of Rodgers Ave.; *ABEFA*, the area removed in preparing grounds around the residences. The great difference in the areas is clearly manifest and after noting this it is logical to conclude that if it required 13 years for the slide to get in motion after this large amount of material was excavated, the removal of the area due to the grading of Rodgers Ave. alone was not the cause of it, as some residents have contended.

Someone may advance the argument that because this material stood for 13 years after its toe was removed that the removal of the toe was not the cause of its movement. This, however, does not logically follow. The forces resisting this movement may have been strained for these years almost to the breaking point, and the mass finally set in motion from some other source. For example, surface water may have found its way to this clay in increasing volume from year to year, and added to its already lubricated state, reducing its coefficient of friction and thereby setting the mass in motion. The entering of the surface water would not then be the fundamental cause, but rather a contributing factor.

RECOMMENDATIONS FOR REMEDIAL WORK

From the foregoing conclusions in regard to the cause of the slide, the following recommendations are made in regard to arresting its further progress. In the opinion of the writer it is not within the scope of the engineering intelligence, with the comparatively meager amount of information now before us, to state unqualifiedly that any one or a combination of the methods suggested will altogether stop this trouble. It would be necessary before arriving at such a conclusion, to sink test holes over the entire area of the slide and drive them to a far greater depth than the two already excavated, but this could be done only at a large expense and it would be wiser economy to apply the expenditure that would be necessary to make such an elaborate system of borings to carrying out the first scheme outlined below, and which should of itself be effective.

The advantages to be named are that it will positively be of some effect in retarding the further movement of the mass, and in carrying it out a better opportunity will be afforded for observing the size and inclination of this vein of fire clay. For it is possible, though not probable, that the vein of clay found in the test hole No. 1 is not the same as found in No. 2, as has been assumed. This is not probable, for if the vein of clay in No. 2 passes under Rodgers Ave. at a lower elevation than No. 1, the slide would be expected to continue on down the hillside to Dakota Ave., instead of terminating on the west side

*The writer does not believe that the water comes from a leaking main, but would suggest that this question be definitely determined by conducting the following simple and inexpensive test. The gate valves of the water main on the outer edges of the slide zone should be closed, as well as all service valves between. A boiler pressure pump, with pressure gage, should then be connected with a fire hydrant in this section and an attempt made to pump up a pressure of 50 or 60 lb. The gage would clearly indicate the presence of any serious leak

of Rodgers Ave. as it does. This vein is thoroughly saturated with water and in this lubricated state the coefficient of friction is very low. It offers, therefore, very little resistance to the propelling force which tends to move this slide, which propelling force is the mass of earth and stone in the hillside above, extending from the foot of the slope in the rear of the residences on Rodgers Ave. to the break in the ground just back of the houses on Summit Ave. This mass acquired its propelling force, or rather put into action the force which was stored up in it, when its supporting toe, the areas indicated on the cross-section, was removed. It is clearly manifest that if it is possible to get below this vein of clay and thoroughly drain it, its state will be changed from a lubricant, offering practically no resistance, to a mass of stiff clay that will offer, in all probability, enough resistance to the propelling force above to arrest further movement of the slide.

First Scheme—The first suggestion is a scheme for draining the vein of clay. The first step would

Fig. 3. Looking North on Rodgers Ave., Bellevue, Penn., Showing Street, Sidewalks and Front Yards Affected by Slide

(Note the retaining walls overturned and slabs of concrete sidewalk forced onto street.)

be to excavate on a line about 15 ft. in the rear of the houses on Rodgers Ave. and parallel with same, shafts 6x6 ft. spaced 30 ft. from center to center throughout the length of the slide, and extending about 2 ft. below the bed of fire clay, which from present information would make the shafts approximately 25 ft. deep. Between these shafts and at an elevation of the bottom of them a tunnel 3 ft. wide by 4 ft. high would be driven. As the tunnel would be driven each way from a shaft, the extreme length of the driving would be only 12 ft., which is an economical length for this class of work. In the bottom of this trench 8-in. terra cotta pipe would be laid, with three-quarter open joint; the quarter joint on the bed of the pipe should be cemented. The terminal of this line on the high end should be extended vertically to the ground surface and be properly capped, enabling the lines to be flushed if it should ever become necessary to do so. The trench or tunnel after the pipe has been properly laid and cemented, should be backfilled with gravel to a height of 4 ft. This gravel should extend through the shafts at the same height, of 4 ft.; or, as an alternative, it might be advisable after the 6x6-ft. shafts have been excavated and the conditions more accurately observed to backfill them, around the pipe, with concrete reinforced with vertical steel rods to a height of 6 ft., thus affording additional anchorage, or resistance to a further movement of the slide.

The measurements given would vary slightly to meet the requirements found after the tunnel was excavated. The object would be to have the gravel extend from 2 ft. below the bottom of the clay to 1 ft. above, with the 8-in. terra cotta pipe at the bottom.

This pipe should have the proper slope to two or more cross drains, which would in turn be connected with the 8-in. sanitary sewer in Rodgers Ave., preferably at a manhole if the manholes are properly located, as it would give an opportunity to observe the flow of water from this drainage system, and if necessary test the working condition of the drainage line. The cross drains, at the option of the contractor, should be laid throughout their length in the same manner as indicated for the main drainage pipe, or as an open ditch. The use of an 8-in. terra cotta pipe is advised not because a pipe of that capacity will be required to carry the water, but because the difference in cost of 4-in. and 8-in. pipe is very small in comparison to the magnitude of the work, and the larger size for many reasons is preferable. The constant flow of water from these drains would be so small that there would be no objection to connecting it with the sanitary sewer. In carrying out this work the cross drains should be constructed first, and carried from the street in to the trunk line; and the trunk line should be constructed each way from a cross drain, thereby insuring drainage during the progress of the work for half of each section, which would be quite a saving in pumping. The cost of such a drainage system based upon the assumption that the shafts will be 25 ft. deep throughout would be approximately as follows:

1000 cu.yd. excavation @ $5	$5000.00
340 cu.yd. gravel delivered at site @ $2.25	765.00
820 lin.ft. 8 in. t.c. pipe delivered at site @ $0.18	147.60
820 lin.ft. 8 in. t.c. pipe laid in place @ $0.10	82.00
	$5994.60

Based on a length of 540 ft. for the trunk-line drain, the cost per linear foot would be $11.10. The price of $5 per cu.yd. for excavation includes the placing of the gravel and backfill, and the cost of tearing up the brick paving in Rodgers Ave. would be more than offset by the excavation for the cross drains not being so deep at that point. This scheme, as outlined, would be cheaper than an open trench, and unless the trench was excavated and backfilled in short sections, there would be grave danger of causing a fresh movement in the whole slide.

Second Scheme—The next scheme, which would be supplemental to the one just outlined, would be to offer additional resistance to the slide by carrying the foundations of the dwellings on the west side of Rodgers Ave., 2 ft. below the vein of clay. This would not be such an expensive proposition as it appears at first thought, as the foundations of practically all of these houses are so badly damaged that they will have to be rebuilt. The cost, therefore, of lowering them 10 or 12 ft. to an elevation below the clay level, would be the cost that should be charged against them as anchors. These walls should be built preferably of concrete, with a few steel reinforc-

ing rods properly placed in the back wall. A rigid box of this size with its bottom extending below the clay into the shale rock and the weight of the house on top, would offer an enormous resistance to movement. Six or eight inches of gravel, forming a French drain, should be placed around the outside of these walls to a height of 4 or 5 ft. from the bottom. In the rear of the houses from these French drains, tunnels should be driven, which would be only 15 ft. long, to the trunk-line drain, placing drain pipe in them laid similar to the trunk line, insuring thereby that all water around the foundations will be carried off.

If the progress of the slide is not arrested by the drainage and anchorage system outlined and it is still desired to stop it, the next move would have to be on broader and more expensive lines.

THIRD SCHEME—The next suggestion would be to remove in a measure the intensity of what has heretofore been termed the propelling force. To do this it would be necessary to start at the break in the ground at the top of the hill just back of Summit Ave. and grade off the side hill on a slope of 1½ horizontal to 1 vertical, extending this slope continuously from the break in the ground at the top of the hill to the level of the rear yards on Rodgers Ave. This would remove a weight of 19,000 tons, or 14,000 cu.yd., which, estimated at 75c. per cu. yd., would cost $10,500, and mean abandoning the alley between Summit and Rodgers Ave.

If the trouble continued after this, a comprehensive set of borings should be made over the whole area of this zone, in order that the location and direction of the vein or veins of water which cause this trouble could be determined beyond a question of doubt, together with a correct knowledge of the underlying strata. In making this investigation the writer would suggest, as a matter of economy, borings instead of excavating test holes. The borings would cost about $1 per linear foot. Having this information at hand the drainage system heretofore outlined could be extended and the water cut off before it reaches the vein of fire clay. With the water removed and the whole vein of underlying clay in this zone changed in character from a lubricant to a material offering resistance to this movement the trouble at this place would cease.

The expense of removing 14,000 cu.yd. of material together with the borings and elaborate drainage system that it might be necessary to construct, would entail an expenditure that would in all probability make it prohibitive. It is, therefore, the recommendation of the writer that the first scheme outlined, together with the foundations of the houses being carried down below the vein of clay, be adopted. The cost of this compared with the subsequent suggestion would be nominal and in the event of it being necessary to resort to the last named scheme the work already done would not be lost.

In rebuilding the retaining walls supporting the terraces on Rodgers Ave., which walls have been overturned by the slide, the foundations should be carried down 5 or 6 ft. below the sidewalk elevation or just below the fire clay, and they should be built of reinforced concrete, properly designed, with the proper proportion of the base extending out in front of the wall under the sidewalk. Broken stone, gravel or a tile drain should be placed back of this wall for its entire length, leading the water collected by same into one of the cross drains of the drainage system.

Silt in the Rio Grande*

BY W. W. FOLLETT†

SYNOPSIS—This report is a valuable addition to the scanty literature on silt in rivers. The abstract printed here gives a brief historical outline of silt determination experiments which were begun in 1889, and with some interruption have been continued to date. This report makes use of data accumulated up to the close of 1912. The methods adopted to determine the weight of a cubic foot of silt deposited in a reservoir under actual conditions and the percentage of silt carried in a cubic foot of water are based on practical experience and judgment rather than on laboratory experiments, as similar determinations have heretofore been made. The conclusions drawn from a careful analysis of the data refer more specifically to the silt accumulations which must be provided against in the Engle Reservoir of the United States Reclamation Service, but the great value of such a collection and analysis of experimental data will be recognized by all hydraulic engineers who have to construct reservoirs on silt-bearing streams.

The Rio Grande is some 1500 miles long, measured along its axis and not counting sinuosities in alluvial valleys. Near its source in Colorado and northern New Mexico, its waters are limpid, carrying practically no sediment in suspension, and are filled with mountain trout.

The whole Rio Grande may be divided as regards its carrying of silt in suspension into three divisions. From its source to Albuquerque, 300 miles, the water is practically clear. From there to the mouth of the Conchos, 500 miles, the water is muddy and is sometimes saturated with silt; that is, is carrying all it possibly can at the rate at which it is moving. From the Conchos to the Gulf of Mexico, 700 miles, it is a silt-bearing stream, but the average percentage of silt is much less than in the middle section.

SILT DETERMINATIONS

HISTORICAL—In 1889 a reservoir was projected in the valley of the Rio Grande just above El Paso, Tex., to be known as the International Reservoir. It was desirable to know how much silt was carried in suspension. Determinations were begun June 10, 1889, and were continued until Aug. 31, 1890. The work was done for the United States Geological Survey under the direction of General (then Major) Anson Mills. Many samples were taken and carefully studied.

Beginning in May, 1897, occasional samples were taken at El Paso until Jan. 1, 1905. On the latter date, systematic observations were begun and were continued at El Paso until May 31, 1910. On May 28, 1905, the taking of samples was begun at San Marcial, 165 miles above El Paso, and has been systematically continued ever since, an average of ten samples being taken each month when the river is flowing.

The El Paso gaging station was taken over from the Geological Survey by the International Boundary Commission, of which General Anson Mills is commissioner and the writer is consulting engineer, in May, 1897, and the San Marcial station was taken over in September, 1900. On Aug. 1, 1910, both stations, as well as all others maintained by the

*Abstract of a special report of the Consulting Engineer of the International Boundary Commission and Advisory Engineer of the Commission for the Equitable Distribution of the Waters of the Rio Grande to the Secretary of State, published as a State Department Document in 1913.

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Boundary Commission, were turned over to the International Water Commission,* of which Wilbur Keblinger is commissioner, and for which the writer acts as consulting engineer, the gaging work being still in his charge.

SILT DETERMINATIONS OF 1889-'90—The first samples were taken from the Rio Grande at El Paso on June 10, 1889. The river was carrying some 4000 cu.ft. sec., and had been of or about that volume for a month, this being the spring flow of snow water. Naturally, the percentage of silt was low. Frequent samples were taken until the river went dry on July 29. It remained dry until Dec. 19, when a flow of about 200 cu.ft. sec. began and continued, with little variation, until Mar. 13, 1890. Then, after a few preliminary flashes, came the spring flood of snow water, which reached a maximum of some 7400 cu.ft. sec. on May 31 and then gradually receded to 250 cu.ft. sec. July 26. Between then and Aug. 31 there were two small rises which carried some silt.

TABLE I. DISCHARGE AND PER CENT. OF SILT IN RIO GRANDE*

Year	Month	Discharge, acre-ft.	Per cent. of silt used in 1890	Per cent. of silt used in this paper	Acre-ft. of silt used in this paper	Acre-ft. of silt used in 1890
1889	June	156,961	0.47	0.55	863	738
	July	14,575	0.20	0.39	57	29
	August	0	0.	0.	0	0
	September	0	0.	0.	0	0
	October	0	0.	0	0	0
	November	0	0	0	0	0
	December	4,366	0.81	0.73	32	35
1890	January	12,054	0.30	0.34	41	36
	February	16,095	0.36	0.38	61	58
	March	26,076	0.61	0.78	203	159
	April	130,305	0.58	0.64	834	756
	May	354,916	0.35	0.34	1207	1242
	June	262,038	0.20	0.22	576	524
	July	52,521	0.13	0.14	74	68
	August	45,141	0.71	0.81	366	320
	Totals	1,075,048			4314	3965

*Made up from several tables in the original.—Ed.

From June 1, 1889, to Aug. 31, 1890, the river carried in suspension past El Paso a quantity of silt equal, when dried out, to the weight of 4314 acre-ft. of water. An experiment described later on in this paper shows that 53 lb. of dry silt will, when wet, occupy 1 cu.ft. of space. Fifty-three pounds is 85% of 62.4 lb., the weight of 1 cu.ft. of water. Hence, 4314 acre-ft. should be divided by 0.85 to get the space which it will occupy, which is 5075 acre-ft.

This sum 5075 divided by 1,075,048, the total discharge of the river in acre-feet, equals 0.47+; or, the average of the silt during the time under consideration was 0.47%. About 90% of the water came in June of 1889 and April, May, June and July of 1890, and, with the exception of a sharp rise June 25, 1889, the rise and fall were gradual. This, together with the season of the year, indicates that the flow was snow water, which is always low in silt. The river was dry during August, September, October and the first half of December, 1889. The first three of these are frequently months of very high silt percentages. The winter flow was small and steady and, as usual, carried a rather low percentage of silt.

When this record was discussed in the Eleventh Annual Report of the Geological Survey, it was assumed that the mean of the determinations made in each month would be the average sediment for the month. This gives just the same weight to a determination, say, of a sample taken Apr. 1, 1890, when the river was carrying about 450 cu.ft. sec. as to one made Apr. 20, when it was carrying about 4100 cu.ft. sec., or nine times as much as on the first. The latter determination should have nine times the weight of the former, as it represents nine times as much discharge.

Table I shows the "sediment ratios" deduced in the Eleventh Annual Report and those deduced in this paper. Both are given in per cent. by weight of water.

It is also assumed in the Eleventh Annual Report that 1 cu.ft. of dry sediment will weigh 100 lb. This may be true, but the vital question is not the weight of a cubic foot of laboratory-dried silt, but how many pounds of dry silt will, when saturated with water, occupy in the bottom of a canal or reservoir 1 cu.ft. of space. This is fully considered later on.

SILT DETERMINATIONS AT EL PASO FROM MAY, 1897, TO DECEMBER, 1904—In May, 1897, the El Paso gaging station was put in charge of the International Boundary Commission. The first sample of water was taken May 7 by the writer under the direction of General Mills.

A "silt trap" was used, it being the same instrument with which the samples had been taken in 1889-'90. It consisted of a brass tube about 2 in. in diameter and 6 in. long with a hinged door on each end and a socket at the side by which it was attached to a long pole. The doors were closed by springs and were held open by a trigger to which a wire was attached.

The doors would be set open, the trap lowered to the desired depth, when the trigger would be sprung by pulling the wire and the doors would close. Then the trap would be pulled up and its contents emptied.

It was an awkward thing to use and did not give good results, as, when the water was very muddy, some mud would always settle before the trap could be emptied and, as the doors were not tight, water would leak out before it could be emptied, and this was probably partially filtered. Shortly after beginning its use, the pole was broken and the trap was lost.

Since the loss of the trap, samples have been taken in a quart bottle. This is weighted and lowered slowly from the surface to the bottom of the river, where a slight pause is made, when it is drawn slowly to the surface. The movement is so timed that the bottle will be filled about the time it reaches the surface. The bottle is tightly corked and so kept until the determination is made.

A considerable number of samples, over a hundred, were taken during 1897-'98 and '99 and were stored in a shed near the gaging station. During the winter of 1899-1900 there came some unusually cold weather and all the samples were frozen. All but five bottles broke and so the samples were lost. During 1900, 1901, 1902, 1903 and 1904, 51 samples were taken.

In June, 1900, the writer arranged with Prof. J. C. Nagle, of the Texas Agricultural and Mechanical College, to determine the percentages of silt of the Rio Grande water. Seven samples taken in 1904 were determined by Arthur W. Houck, an assayer and chemist residing in El Paso.

Prof. Nagle obtained percentages by volume by putting some of the muddy water in a long test tube and then letting it stand 7 days. He then measured the depth of the clear water and of mud and so obtained his relation. The percentages thus obtained were from three to four times as large as those by weight, dry, and were discarded as having no relation to actual conditions in a reservoir or canal.

DETERMINATION OF THE WEIGHT OF A CUBIC FOOT OF SILT

In 1904 the Reclamation Service began a careful study of the Rio Grande conditions, preparatory to the development of the Rio Grande project. B. M. Hall was the supervising engineer in charge, and he discussed with the writer the silt problem in all its phases. The question of paramount importance was the space which the silt would occupy in the proposed reservoir. It was evident that the per cent. by weight, dry, was too small, as the silt would never be dry in the bottom of a reservoir; and it is a well known fact that clay or clayey earth will swell when wet, and that all Rio Grande silt contains a large percentage of clay.

This statement is based on the assumption that the specific gravity of the dry silt is the same as that of water. While, at first sight this seems improbable, it appearing that its weight would be more than that of water, the investigators knew that the Rio Grande silt was extremely light. Others since 1904 have assumed that a cubic foot dry weighs 85 lb., but there is no statement given of an actual weighing of a fixed quantity of dry silt, no more than there was in 1890, when it was assumed to weigh 100 lb. per cu.ft.

It was not the specific gravity of dry silt which was wanted anyway. It cut no figure whatever in the problem. The unit which was desired was the amount of silt which would fill a cubic foot of space when fully settled and when compressed by the overlying load of water, and its relation to the per cent. by weight of dry silt to the water which carried it.

It was evident that the per cent. by bulk, obtained from test tubes, would be too large for the desired unit because there was no weight on the silt in the tube to compact it, as there would be in a reservoir, and because the mud would stick to the glass and so not settle.

Something more than guess work was wanted. It did not seem proper to us to found all our silt calculations on an assumed bulk for the silt which was, as it were, simply pulled out of the air. The desire was to approximate as closely as possible to the conditions which would be found in the bottom of a reservoir.

After considering various schemes, to all of which there seemed to be valid objections, it was finally decided to seek a mud bar in the river where the water had been comparatively still and which had shrunk enough to show material cracks, and to cut from this bar a 3-in. cube, have it dried out and weighed and to abide by the result, whatever it might be.

The idea was that a bar should be chosen which had shrunken enough to make up for the compression which the silt in the bottom of a reservoir would undergo from the weight of the water over it. Of course, the necessary

*Commission for the Equitable Distribution of the Waters of the Rio Grande.

amount of shrinkage could not be told exactly, but it was thought that a fairly good guess could be made.

On June 29, 1904, Mr. Hall and the writer carefully cut a 3-in. cube of silt free from gravel out of a sediment bar just above the Mexican dam at El Paso. The bar showed signs of material shrinkage since the water had receded, there being large cracks in it, so that it was decided that the piece cut out, having already shrunken considerably from exposure to the air, would fairly meet the prescribed requirements

This sample was taken to Arthur W. Houck, who had had some 15 years' experience in analytical chemical work. He dried it out at a temperature high enough to evaporate the water, but not so high as to volatilize any of its substance, and then weighed the residual. He found its dry weight to be 5789 gr. or 0.827 lb. (7000 gr. to the pound). There would be 64 of these cubes in a cubic foot, or a cubic foot of wet silt in the bar from which this was taken, when dried out, would weigh 53 lb. or 85% as much as a cubic foot of water (the latter weighing 62.4 lb.).

It has been assumed that the above experiment fairly determined the weight of reservoir silt and that all silt determinations made in 1889-'90 should be divided by 0.85 in order to obtain the actual final volume of the silt.

It may be objected that the results involved are too important to make them depend upon one single observation, as was done in this case. However, the conditions under which the silt will settle in a reservoir are not well known, and there are enough other indeterminate factors in the problem to render it probable that this one sample, deliberately and intelligently chosen by two trained men who were fully conversant with all the conditions confronting them and who were careful in their selection of the sample, would give a result fully as good as might come from the mean of many samples taken with less care. Moreover, the result appeared to be reasonable.

The Rio Grande silt contains no sand coarse enough to be perceptible. It is composed of finely dessicated and almost impalpable clay and alluvium and it settles slowly. Therefore, it is not proper to compare its density when settled with that of silt which has much sand in it. Of course, the suspended silt, when settling in the river, frequently becomes mixed with the sand which is moving on the bottom of the river, and hence mud bars frequently show some sand. This sand was not carried in suspension, however.

SILT DETERMINATIONS AT EL PASO FROM JAN. 1, 1905, TO MAY 31, 1910

Beginning Jan. 1, 1905, samples were taken more frequently at El Paso. For the first few months, four or five were taken each month; then one was taken every time a meter measurement of the river was made. This was continued until May 31, 1910; 608 samples in all were taken.

Samples Nos. 1 to 5 inclusive, were, as previously stated, determined by volume only by Prof. J. C. Nagle. Nos. 6 to 49 inclusive, were determined for both weight and volume by Prof. Nagle. From these last, the proper weight determinations for the first five were computed. Samples Nos. 50 to 56 inclusive were determined by Arthur W. Houck. Samples Nos. 57 to 304 inclusive, being those taken between Jan. 1, 1905, and April 30, 1907, were determined at the Berkeley, Calif., laboratory of the Reclamation Service. Samples 305 to 608 were determined by Mr. Houck in his laboratory at El Paso.

SILT CARRIED IN SUSPENSION PAST SAN MARCIAL FROM 1897 to 1912

San Marcial is some 40 miles above the Engle Reservoir of the Rio Grande project of the Reclamation Service. The greater part, if not all, of the silt passing San Marcial in suspension will ultimately find lodgment in the reservoir.

After the study of the El Paso reservoir was completed, a similar examination was made of the Engle reservoir. In making this, percentages for San Marcial discharge were needed. The El Paso table of monthly percentages was used, with some modifications which were made necessary by the stream's flow at the two stations. Subsequent study indicates that there were not changes enough made in the El Paso table and also that the percentages used therein for storm-water months were usually too small, while that for usual "winter flow" was too large.

SILT PERCENTAGES FOR 1905 TO 1912—Samples for San Marcial water were not taken prior to June, 1905. For June, July and August a guess was made, based on the determinations of the 27 mixed samples. From September on, the monthly means were deduced from the determination for each month.

October, 1911, is typical of months of sudden fluctuations in flow and of a large percentage of silt. The sudden increase in per cent. of silt on Oct. 7 from 4.52% to 14.87% was coincident with an increase in discharge from 79[illegible] to 11,780 cu.ft. sec. Such sudden increases in flow during [illegible] summer are usually accompanied by a jump in the silt.

This record for October is also instructive in other [illegible] and can be profitably analyzed at some length. It was [illegible] that a large increase of flow on the 6th was accompanied by a decrease in silt instead of an increase, and that, between the 16th and 19th, the silt dropped tremendously, while the flow decreased only 40%. The explanation of this probably is that there were general rains over the whole Rio Grande watershed.

In the latter part of September a small Puerco (a West[illegible] branch of the Rio Grande) rise brought in some silt, but the rise on Oct. 6 was from the headwaters. On Oct. 7 a big Puerco and Salado (a branch south of the Puerco) silt-bearing flood mixed with the headwaters flow. Between the 16th and 19th, the Puerco flood ran entirely out, while a good flow continued from above, and the silt dropped accordingly. The sum of the 13 determinations used during October, divided by 13, gives 5.37% as the mean for the month, while the actual mean is 7.90%. This shows the error which may arise from mixing samples.

SUMMARY—Table II shows the total discharge in acre-feet of water, the mean per cent. of silt and the total silt in suspension for the 16 years at San Marcial.

TABLE II. TOTAL ACRE-FEET OF SILT WHICH HAS PASSED SAN MARCIAL, N. M., IN SUSPENSION FROM 1897 to 1912, INCLUSIVE

Year	Acre-ft. of water	Per cent. of silt	Acre-ft. of silt
1897	2,215,953	1.72—	38,051
8	960,981	1.55—	14,858
9	239,434	2.14+	5,127
1900	467,703	2.02+	9,459
1	656,252	2.82	18,603
2	200,729	3.05	6,123
3	1,272,069	0.97—	12,319
4	709,796	2.37+	16,838
5	2,422,008	0.78—	18,875
6	1,563,737	0.89+	13,901
7	2,157,709	1.11—	23,889
8	774,109	2.00—	15,469
9	1,279,934	1.51—	19,318
1910	852,092	0.76+	6,520
11	1,799,733	4.14+	74,563
12	1,499,614	1.47—	22,018
Totals	19,072,453		315,831
Per year	1,192,028	1.66—	19,739

Table II shows an annual percentage varying all the way from 0.76 to 4.14%. A study of the table, omitting 1911, would indicate, in a general way, the larger the discharge, the smaller was likely to be the per cent. of silt, so that the absolute amount in acre-feet of silt each year did not vary as much as did the discharge. But the 1911 record completely overthrows this theory and shows that the only way to keep up with the silt is to keep measuring it.

The silt for 1911 was normal for ten of the twelve months. But July and October were months of large flow, with sudden floods, when the percentage of silt increased enormously. The two months furnished 40% of the annual discharge and carried 85% of the silt for the year. The 1911 silt was practically twice that of the next heaviest year (1897) when over 400,000 acre-feet more water passed San Marcial than in 1911.

COMPARISON OF PERCENTAGES, SAN MARCIAL AND EL PASO

During the four years, 1906 to 1909, silt was determined at both El Paso and San Marcial. During those years 5,775,000 acre-ft. of water carried past San Marcial 72,580 acre-ft. of silt, showing a mean percentage of 1.26, while 4,291,000 acre-ft. of water carried past El Paso 61,820 acre-ft. of silt, showing a mean percentage of 1.44. This shows an excess at El Paso of 0.18% of silt over San Marcial. Assuming that this difference existed from Jan. 1, 1897, to May 31, 1905, this covering the time when the El Paso percentage was used at San Marcial, there passed San Marcial during this time 8,285,000 acre-ft. of water, which, multiplied by 0.18 equals 14,913 acre-ft. This indicates that the silt at San Marcial is overestimated for this period 14,900 acre-ft.

UNUSUAL WINTER FLOW—Subsequent determinations show that the percentage adopted in 1905 for the "usual winter flow" (1.06%) was too high. The computation made at that time used all the winter samples, giving equal weight to each of the four groups. An examination of the subsequent determinations of months of similar flow would indicate that about 0.60 would have been nearer right, making this assumption of 1.06 about 0.45 in excess.

Examination of the first eight years shows that this percentage (1.06) was applied to 759,000 acre-ft. of water, which passed San Marcial in November, December, January, February or March. If this percentage were in excess 0.45 then the estimate of silt for these months is 3420 acre-ft. too large.

SUMMER MONTHS OF SUDDEN RISES—During the years 1897 to 1904, inclusive, there were 19 summer and fall months which had from one to three typical summer rises. Only one of these, that for September, 1900, was properly sampled and the El Paso mean for that month was 5.81%. Two samples were taken in August, 1904, which averaged 7.40% and one was taken in October, 1904, which showed 8.09%. These last two appeared so large in 1905 that they were then considered abnormal and were rejected.

An examination of the daily record at San Marcial from 1897 to 1904 and each month entered therein showed typical sudden summer rises such as usually carry much silt. The totals show that the 19 summer and fall months listed had a total discharge of 1,492,000 acre-ft. of water and were credited with 53,550 acre-ft. of silt, or an average of 3.59%. In order to see whether this percentage was too large or too small, the record was similarly examined for the years 1905 to 1912 and 22 summer and fall months were found which had similar sudden rises.

The totals show that the 22 months listed had a total discharge of 1,841,525 acre-ft. of water and carried in suspension 111,193 acre-ft. of silt, or an average of 6.04%. The difference between 6.04 and 3.59 is 2.45. If the average silt for the months of sudden summer rises prior to 1905 was the same as it was subsequent to that year, then the San Marcial silt has been underestimated 2.45% on 1,492,000 acre-ft. of water, or 36,550 acre-ft.

SUMMARY OF COMPARISON—The account then stands:

	Acre-ft.
Over-estimate on winter flow	3,420
Over-estimate on percentage	14,900
Total over-estimate	18,320
Under-estimate on summer months	36,550
Net under-estimate	18,230

SEDIMENT MOVING ON BOTTOM OF RIVER

There is also a large amount of sediment which is rolled along the bottom. No method has yet been devised of measuring this moving sediment. That its amount is large is shown indirectly by many phenomena. How large it is the writer is unable to say with any degree of confidence. That it may amount to 25% of the silt carried in suspension he believes possible.

The amount of bottom sediment will vary greatly with different streams, the factors affecting it being the weight and fineness of the sediment, the declivity of the stream's bed and the number and abruptness of variations in flow. It is unfortunate that it cannot be measured and studied as can the silt in suspension. It adds an indeterminate factor to the problem which is troublesomely large.

The only way its amount can ever be arrived at will be by the measuring, for a long series of years, of the deposit in some existing reservoir, together with the delta at its head, while silt determinations are kept up. In this way a result may finally be deduced for one stream, but the question of its applicability to other streams will be an open one.

STREAM FLOW

From 1897 to 1899, inclusive, the daily discharges are taken from the records of the Geological Survey. Twenty-one meter measurements were taken in 1897, fairly well distributed, but not catching the high point of the May flood. Only two measurements were taken in 1898, and eight in 1899.

As the cross-section is a rapidly changing one, the computed discharges for 1898 and 1899 may be largely in error, and those for 1897 are likely to be considerably so. It is possible that this error, for each month, does not exceed 25% and the table for each year may not be in error this much. During 1900, the station was operated partly by the Geological Survey and partly by the Boundary Commission and the record is somewhat better, although it may be 15% in error.

From 1901 to 1912, systematic meter measurements have been kept up. Ten are made each normal month. During periods of floods, more are taken, the instructions to the observer making him sure to catch all material fluctuations in flow. The gage has always been read twice daily.

While a single measurement may be 6 or 8% in error, the errors of one will tend to balance those of another, so that the mean error of meter measurements is believed to be considerably less than 5%. This is not a guess, but is based on ten years' study and comparison of thousands of measurements. The interpolations of discharge for the days when no measurement was taken are believed to be not over 5% in error.

SILT SAMPLING AND DETERMINATION

The samples of water were sometimes taken in the swiftest place and sometimes in slower water. The section was integrated by the slow lowering and raising of the bottle and the sample probably showed the proper percentage for the particular place in which it was taken within a small margin of error. How near it came to giving the percentage for the whole section is problematical. Occasionally, a sample might be 20% or 25% in error. As in the discharge, however, an error one day might be partially or wholly balanced by an error in the other direction another day. It is probable that the silt samples average not over 10% in error, and possibly that they are not 5% in error.

The writer does not know by what process the determinations were made by Prof. Nagle or by the Berkeley laboratory, but it is fair to assume that good results were obtained. Mr. Houck first weighs carefully the whole sample. Then he runs it all through filter paper, dries out and weighs the residual, and obtains his per cent. from the two weights. The process is accurate.

WEIGHT OF SILT—The most perplexing question involved is what space the silt will ultimately occupy in the bottom of the reservoir where it is wet the greater part of the time; where it will shrink some on exposure to the air, but swell again when the water covers it, and where it is compressed by the water over it. It was only after the most careful analysis of all these conditions that Mr. Hall and the writer took the samples described in the beginning of this article and adopted its dry weight as the weight of the silt which would occupy a 3-in. cube in the bottom of a reservoir. The bar from which this was taken had been exposed to the air for several months. The sample was taken June 29, 1904, and the record shows that the river had been dry since Mar. 1 and had practically been dry since Feb. 7. The bar had large cracks in it and it was assumed that the shrinkage shown by these cracks would equal the compression in the bottom of the reservoir due to the superincumbent load.

It has always been the writer's custom when approaching a problem to study it carefully, weigh it in all its bearings, assume the premises which seem most probable and then abide by the result, whatever it may be, so long as it is a reasonable one. This was done in this case and the result of 53 lb. of dry silt to 1 cu.ft. of space filled was adopted. This unit is derived from experiment, but so carefully derived and so probable does it seem that it should stand until some actual measurements of sediment in a reservoir disprove it. If in error at all, it is on the safe side. Each reader must decide for himself what percentage of error is involved.

NET RESULTS

It is believed that the reservoir capacity (315,831 acre-ft.) the silt past San Marcial for the whole 16 years, 1897 to 1912, would occupy is not in error over 15% either way. That is to say, that somewhere between 270,000 and 360,000 acre-ft. of reservoir capacity would have been absorbed. The error may not be over 10%, in which case the range would be between 285,000 and 347,000 acre-ft.

In this connection, it is interesting to note that the Reclamation Service has tentatively adopted for their service table of the Engle Reservoir a flat rate of 1.8% of silt. This would give, for the 16 years under consideration, 342,000 acre-ft. The writer understands that the use of this percentage is tentative only and may be changed when further knowledge is had of the settlement of silt under water. The fact is here noted as confirmatory of the results given in this paper.

EFFECT ON ENGLE RESERVOIR

This silt, together with that indeterminate amount of sediment which moves along the bottom, has passed San Marcial during the last 16 years, and it has moved toward the Engle Reservoir. The study of what effect it will have on the reservoir and of what should be done to handle it, is outside the scope of this paper. Suffice to say that the engineers of the Reclamation Service are fully cognizant of the conditions and are engaged in devising some scheme which will most expeditiously and cheaply transport the silt, or a large percentage of it, through the reservoir.

Oilproof Concrete can be obtained by coating the surface of the concrete with a mixture of ordinary glue and a 5% porassium bichromate solution, according to some limited experiments made by J. L. Jeffery, published in "Concrete and Construcional Engineering" for December, 1913. Mr. Jeffery made a number of experiments with glue, gelatin and other chemical coatings, but found that the mixture noted was the only one in the lot that he tried which would prevent the oil from soaking into and out of the concrete. He says that lack of time prevented him from carrying out experiments on the same lines with the heavier oils or with crude petroleum. The tests were made with kerosene. He thinks, however, that no greater difficulties would result from the use of heavy oils than from the more volatile kerosene used in the experiments described.

The Pittsburgh Sewer Explosion

By N. S. Sprague* and Chas. M. Reppert†

On Nov. 25, 1913, a serious sewer explosion occurred in Pittsburgh, which is believed by the authors to be the most extensive and disastrous on record. Owing to the increasing frequency of sewer explosions in recent years in different sections of the country, the subject deserves careful consideration and study in order to determine the causes, with a view to adopting preventive measures. The damage wrought by the recent explosion in this city and the consequent clogging of an 8-ft. trunk sewer serving a population of 50,000, with an ordinary dry-weather discharge of 15 million gallons daily, presented a serious situation. Protection of property from flooding, maintenance of traffic in streets and by railroad, continuance of operation of large industrial plants, and safe-

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Fig. 1. Effects of Explosion on Sassafras St. (Nov. 25, 1913)
(Looking west toward brewery. Ditch afterward settled over sewer.)

Fig. 2. Damage at Bottling House, Pittsburgh Brewing Co. (Nov. 26, 1913)

Fig. 3. Blowout near Forfar St. Bridge; Sewage from Upper Section Flowing into Break, Nov. 28, 1913

Fig. 4. The Same Blowout Secured by Steel Sheeting (Dec. 6) after Caving Had Blocked Sewer and Undermined Crane

Four Views of the Great Sewer Explosion in Pittsburgh

guarding the public health, demanded prompt and effective action.

The problem confronting the city immediately after the explosion was to provide a temporary channel for the sewage and the repair of the damaged sewer. An account of how the emergency was met, together with a description of the effects of the explosion, is the purpose of this article.

Description of Sewer and Territory Served

The sewer in which the explosion occurred is the main trunk sewer of a watershed comprising an area of 1700 acres, known as the 33rd St. or Two Mile Run Drainage Basin. This watershed is located in the old city between the rivers, the outlet of its trunk sewer discharging into the Allegheny River about two miles above the "Point." The basin includes portions of several of the older residential districts of the city (Shadyside, East Liberty, Bloomfield and Lawrenceville). Along the line of the sewer there are located in the valley a number of industrial plants, and the vicinity of the outlet is exclusively a manufacturing district.

The sewerage system of the basin is on the combined plan. The territory embraced within the watershed is practically all improved and sewered, and has a population slightly in excess of 50,000. The trunk sewer follows the line of a former water-course located for the most part in a valley with rather steep slopes. It extends up 33rd St. from the river a short distance above Liberty Ave., where, turning sharply to the east, it continues up a ravine nearly parallel with the Pittsburgh Junction R.R. and the Pennsylvania R.R., the latter being at a somewhat higher elevation. The sewer on leaving 33rd St. crosses under the right-of-way of the Pittsburgh Junction R.R., thence by a straight line across private property of the Pittsburgh Brewing Co. to Sassafras St., extending to the vicinity of 39th St. Beyond 39th St., the sewer again crosses private property, going under a number of buildings and manufacturing plants, and then follows the line of Neville St., an unimproved thoroughfare bordering the right-of-way of the Pittsburgh Junction R.R., and continues on this street up the valley. The sewer above Liberty Ave. is hemmed in by buildings and railroads on one side and a very steep hillside on the other.

Both sides of 33rd St., from the river to Penn Ave., are occupied by the Upper Union Mills of the Carnegie Steel Co., while between 33rd and Cayuga Sts., the upper limit of the explosion, the sewer passes under buildings of the Pittsburgh Brewing Co., the Pittsburgh Rivet Co., Pittsburgh Cold Rolled Steel Co., the Petroleum Products Co., and two frame dwellings on Neville St. The location of the sewer in the ravine is remote from built-up sections; steeply inclined hillsides intervene, and the lateral connections of the sewer extend on sharp grades up the hillsides to the streets lying beyond. The Pittsburgh Junction R.R., which parallels the sewer, crosses Liberty Ave. at grade and is carried down 33rd St. toward the river on a viaduct supported by columns at the curb line. The lower end of the basin, below Liberty Ave., is comparatively level.

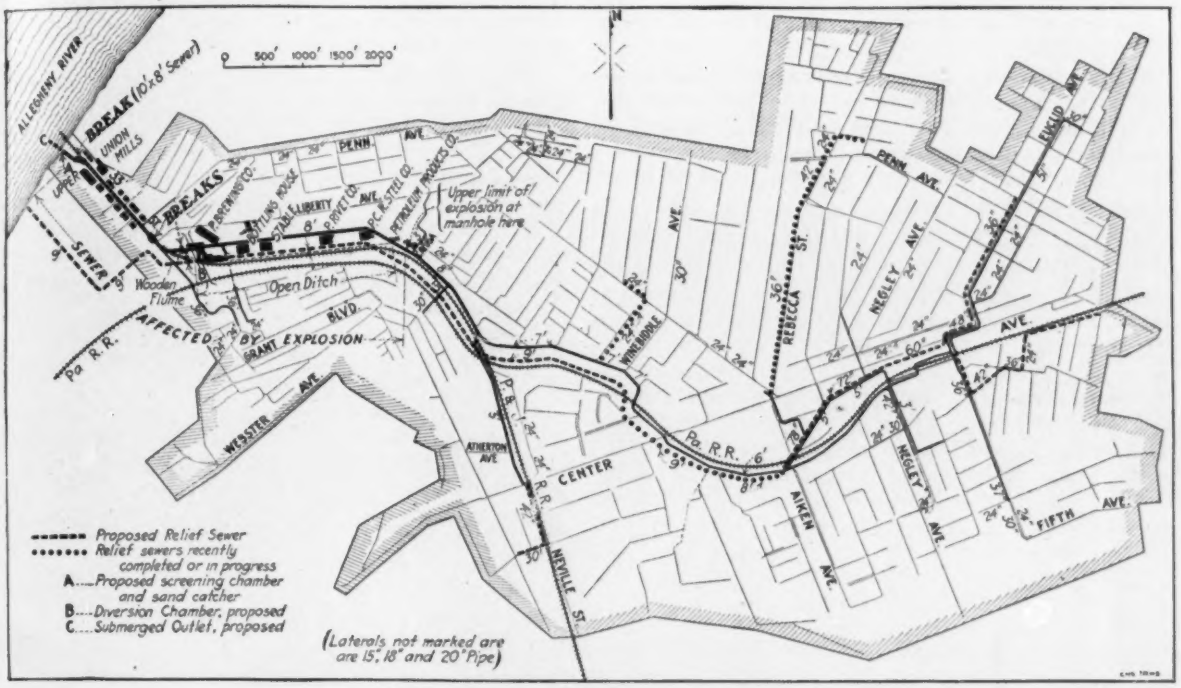

Fig. 5. Drainage Area of 33d St. Trunk Sewer, Pittsburgh

(The explosion of Nov. 25, 1913, affected the lower 5300-ft. length of sewer, from the river to a manhole near Cayuga St.)

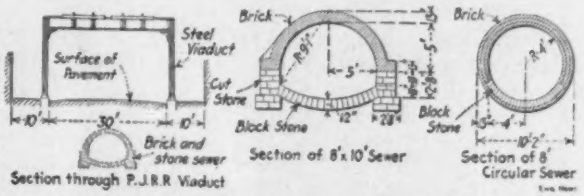

Fig. 6. Sections of 33d St. Trunk Sewer

The section of the sewer affected by the explosion extends from the outlet for a distance of approximately 5300 ft. to a point near the intersection of Neville and Cayuga Sts. From the outlet to Penn. Ave. the sewer is 10 ft. in width and 8 ft. in height, of brick and stone construction with the exception of a short section of concrete sewer immediately above the outlet, which is an extension of the original sewer. Beyond Penn Ave., the sewer is circular in section, 8 ft. in diameter, and is constructed of three rings of brick, the invert being lined with block stone. The 8-ft. sewer was built in 1891. The hydraulic grade of the sewer is about 10 ft. per 1000; the capacity is estimated at 800 cu.ft. per sec.

Possibly 90% of the public garages of the city are located within the watershed. There are also a number of miscellaneous storage houses, dry-cleaning establishments, and paint-manufacturing shops. The acid waste discharged into the sewer includes the discharge from pickling vats of steel companies. A plant of the Pintsch Compressing Co. is also located near the line of the sewer, at Cayuga and Neville Sts.

The Effects of the Explosion

The explosion occurred about 2:30 p.m. The immediate effects indicated an explosion of great violence, affecting 5300 lin.ft. of the trunk sewer and extending some distance along the lateral connections. Street pavements were heaved, ruptured and torn asunder; manhole lids were blown high in the air; the brick masonry of the manholes was shattered and in some cases demolished; window glass was broken for some distance from the line of the sewer; sections of the sewer collapsed, stopping the flow and causing the street to cave in. Buildings and industrial plants over or in close proximity to the line of the sewer were partially wrecked or damaged, either by the explosion itself or by subsequent settlement. The location of the sewer, the depth of cover, and the topography of the vicinity fortunately operated to confine most of the damage to the sewer itself and the overlying roadways, thus reducing the extent of the damage to private property and the interference to traffic and business.

At the time of the explosion the outlet of the sewer was submerged to within 1½ ft. of the top, so that the flow was backed up for several hundred feet. The explosion expelled the sewage from the outlet with terrific force.

Damage to Sewer—At a point 220 ft. above the outlet about 20 lin.ft. of the sewer arch immediately collapsed, an opening about 25 ft. square at the surface of the ground resulting; and the arch of about 100 lin.ft. of adjoining sewer was ruptured, requiring its removal. The break occurred at a point where the sewer formerly discharged into the river and where the sewer section changed from brick to concrete. The connection between the two types of construction was probably a plane of weakness. The depth of cover at this break was 14 ft.

Practically all of the manholes and catchbasins in the district below Liberty Ave. were damaged or destroyed, those on the main sewer being the most affected. From the break at the river to Mulberry Alley, where the cover over the sewer decreases to 4 ft., the sewer was undamaged except at manholes. The blockstone paving of 33rd St., from Mulberry Alley to Penn Ave., was heaved and ruptured directly over the sewer and the arch of the sewer itself was split.

Between Penn and Liberty Aves., a distance of 21[illegible] ft., where the cover ranges from 3 to 4½ ft., the effects of the explosion were more severe. Near the abutment of the viaduct at Liberty Ave., the arch of the sewer was demolished and the ground caved in immediately after the explosion, the opening being about 48 ft. long. The arch of the balance of the sewer between Liberty and Penn Aves. was ruptured to such an extent that immediate uncovering of the sewer was made necessary. A ridge several inches in height in the cobblestone paving indicated that the ground over the sewer was raised bodily and split open directly over the crown, the ridge being left as the ground settled back into place. The cast-iron lid of a manhole near the abutment was blown up through the floor of the viaduct above.

Beyond the north line of Liberty Ave. to the manhole near the Forfar St. bridge, a distance of 480 ft., the sewer is in deep cut, the depth to the crown ranging from 19 to 15 ft. There were no surface indications of the sewer's being damaged between these points, except an extensive opening at the manhole near the bridge. In the vicinity of this manhole for a considerable distance the sewer collapsed and the surrounding ground caved in, opening up a large hole which gradually extended as the settling of the sewer arch continued. A manhole on 33d St. just above Liberty Ave. was shattered, the brickwork (of excellent quality) being cracked and ruptured from the top of the sewer to the surface of the street. The cast-iron cover of this manhole was found 150 ft. away in a horizontal position, embedded an inch in the brick sidewalk. Subsequent investigations showed that the arch of the sewer was either broken or ruptured along the crown from Liberty Ave. to the Forfar St. bridge.

Surface settlement over the sewer where located on private property of the brewery, between 33d and Sassafras Sts., indicates that the sewer has been more or less damaged, this opinion being verified by an examination at a shaft near Sassafras St.

From the point where the sewer enters Sassafras St. to the end of the explosion, a distance of 2600 ft., the arch of the sewer was blown off, causing the sewer to choke and the pavement and street surface to settle.

Sassafras St. was paved with blockstone on a gravel foundation to 38th St. The street is 40 ft. wide and has an 8-ft. sidewalk on the southerly side. The sewer is located 10 ft. from the south line of the street. The pavement and curb were blown up and rapidly settled, causing a deep depression over the sewer.

Exterior Damage—The bottling house of the Pittsburgh Brewing Co., a substantial two-story brick building, is parallel to and 20 ft. from the center line of the sewer. The stone masonry of the foundation of this building, which extends several feet above the surface of the ground, was badly spalled and the wall was pushed out of line toward the sewer several inches at the eastern end. Some idea of the force of the explosion is conveyed when it is stated that this building was apparently lifted off its foundation, the location of the sewer being 20 ft. distant and 12 to 15 ft. below the street level.

Between 38th and Cayuga Sts. the sewer, as previously stated, passes in close proximity to (or under) a number of manufacturing plants and frame dwellings.

The plant of the Pittsburgh Rivet Co., a steel-frame mill building with wooden sides, located on Sassafras St., standing 10 ft. from the center of the sewer, was dam-

aged by the explosion. The side next to the sewer was blown from its foundation and a 6000-gal. water tank was moved laterally about 8 in.

Just beyond the plant of the Pittsburgh Rivet Co. is located the steel frame mill building of the Pittsburgh Cold Rolled Steel Co. One of the furnaces of this plant is located partly over the sewer. The explosion entirely wrecked these furnaces and stack, twisted the corrugated-iron sheeting of the building, and damaged the roof. Adjoining this plant is the plant of the Petroleum Products Co. The sewer passes directly under this property and also under and close to a number of large gasoline-storage tanks. A 12,000-gal. gasoline tank, about 10 ft. in diameter and weighing between 24 and 25 tons at the time of the explosion, was lifted about 3½ ft. from its foundation; adjacent tanks were also disturbed.

A freight car standing on a siding directly over the sewer was blown from the track, landing on its wheels at a slight skew with the track.

tire collapse of a long section made it necessary to clear away the obstruction immediately and to devise a plan for bypassing the drainage around the collapsed section on Sassafras St. in order to prevent flooding and consequent damage to property. Ordinances were passed by Council approximating $50,000 and authorizing the letting of an emergency contract on a force account basis.

THE EMERGENCY CONTRACT—The work included under this contract consisted principally of uncovering the sewer, sheeting and bracing the trench, excavating an open channel, cleaning out the invert of the damaged sewer and other incidental work. The essential features of the contract were as follows:

1. The contractor is required to furnish an adequate force, plant and equipment.
2. Labor, materials and supplies, dumpage charges, delivery and removal of equipment, etc., are paid for by the city at actual cost plus 15% for profit and overhead charges. Small tools are furnished without charge.
3. The maximum rates for various classes of work and the rental prices for equipment are stipulated.

FIGS. 7 AND 8. WORK IN LOWER SECTION OF EXPLODED SEWER

(Fig. 7. Operations at break near river. Opening has been excavated, sheeted and braced. Removal of damaged sewer and débris in progress, Dec. 5, 1913.)

(Fig. 8. Sheeted excavation down to sewer, between Penn Ave. and Liberty Ave., Dec. 8, 1913.)

Two dilapidated frame dwellings which were unoccupied were totally wrecked.

Beyond the Petroleum Products Co. there is no paving over the sewer. A ditch quickly formed, due to the collapse of the sewer.

Fortunately, only relatively small gas and water pipes were laid parallel to the sewer. These lines were broken. Large water mains (42-, 36- and 30-in.) cross over the sewer at Smallman St., Penn Ave. and Liberty Ave., respectively, and none of these pipes was damaged.

INVERT UNDAMAGED—Inspections made after uncovering the sewer show that only the arch above the springing line has been damaged. The invert apparently has not been affected. In most places the sewer is found to be split along the crown of the arch, the arch flattened and in some places destroyed entirely.

THE QUESTION OF CAUSE—The causes of the explosion have not yet been determined, and a statement as to cause cannot be presented until the facts have been ascertained by thorough investigation.

EMERGENCY OPERATIONS

The breaks in the sewer at certain places and the en-

4. Transportation of excavated materials by freight is paid for at actual cost.
5. The city assumes the cost of liability insurance without additional profit, and is responsible for damage or loss of equipment with restrictions as to negligence and defects.
6. Provision is made for furnishing additional labor and equipment at prices to be agreed upon in writing.
7. The books of the contractor are open to audit by the city and the contractor is required to furnish full information daily as to the cost, etc.
8. The contract may be terminated by the city on three days' notice.

This contract was awarded to John F. Casey Co., of Pittsburgh, the night of the explosion. This contractor had plant and an organization at work on a near-by job, which was readily obtainable and which made it possible to start the emergency work at 7 a.m., Nov. 26. Since the time of starting, a force ranging from 400 to 500 men has been continuously employed day and night.

CLEARING THE OUTLET—The break in the sewer near the river obstructed the flow to such an extent that there was imminent danger of the sewer becoming entirely clogged. Had this happened the steam-supply lines to the Carnegie mills would have been submerged, causing the mills to suspend operations and so throwing 2000

employees out of work. Furthermore, the overflowing of the sewer at this point would have made a channel to the river, undermining the railroad tracks and stopping the operation of the road.

The sewer was unobstructed beyond the break. In order to clear the obstruction at the break a shaft was excavated directly below, to the top of the sewer. An opening was made in the sewer inside the shaft with a 3½-ton drop hammer operated from a locomotive crane. This opening released the water, which had backed up to within a few inches of the steam-supply line and about 1½ ft. from the street surface. The excavation was then continued, and by Nov. 29, the sewer was uncovered and the trench sheeted and braced for a distance of about 50 ft. Subsequently the trench was extended and portions of the damaged arch of the sewer and débris were removed and an unobstructed flow secured.

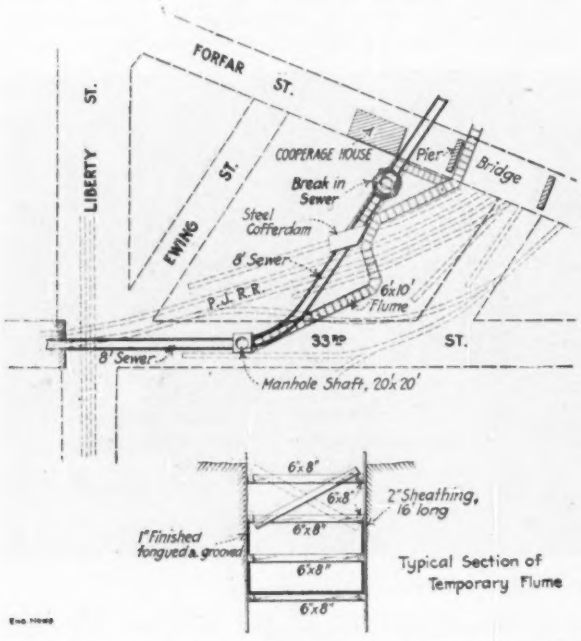

FIG. 9. FLUME DIVERSION OF SEWER JUST ABOVE LIBERTY AVE.

The length of the trench at this point is 86 ft., the width 16 ft. and the depth to the top of the sewer 14 ft. The trench was sheeted with 2-in. lumber, with 8x8-in. wales and 6x8-in. cross-braces. To avoid imminent danger of flooding in case of rain, it is proposed to let a contract immediately for the permanent repairs, at an estimated cost of $7500; this is easily possible because the repair work at this point is separated from the main work.

CLEARING INTERMEDIATE SECTION—Work was commenced Nov. 26, uncovering the damaged sewer between Liberty and Penn Aves., and by Nov. 29, the entire section, 220 ft. long, had been excavated and the sheeting driven to the top of the sewer. Since then the excavation, sheeting and bracing of the trench and the removal of the damaged arch and débris in the sewer have been completed. Owing to the presence of the railway viaduct the work at this point was done entirely by hand.

BYPASSING UPPER PART OF SEWER—At Cayuga St., the upper end of the explosion, the sewer was effectually closed by the settlement of the ground into the sewer. The sewage overflowed and followed down the right-of-way of the Pittsburgh Junction R.R., submerging the tracks and damaging the roadbed. It reëntered the sewer at the opening caused by the break near the Forfar St. bridge. The flow into the sewer was partially obstructed, causing the sewage to back up, but the obstruction was not sufficient to divert the flow to Liberty Ave. Had the sewer clogged entirely at this point serious damage would have resulted to property on Penn and Liberty Aves., below this point.

An attempt was made on Nov. 27, to free this opening of obstructions by excavating with an orange-peel bucket operated by a locomotive crane. After working several hours and excavating to a depth of about 18 ft., the sides of the excavation began to cave in, threatening to undermine the track and endanger the crane, compelling the suspension of this work. To safely continue the work it was necessary to support the track upon which to operate the equipment, and pile bents were constructed. The track having been made secure, a coffer-dam of 12½-in. U. S. steel sheet-piling was driven around the opening. This coffer-dam was about 40 ft. long and 20 ft. wide, the piles being 30 ft. long. The prompt delivery of the piles was made possible by the courtesy of the steel mills in changing the rolls and immediately rolling the material, which was delivered upon the work within 12 hr. Driving the sheeting was started on the night of Nov. 29, and finished Dec. 2. By the time the sheet-piling had been driven, the opening into the sewer had become entirely closed. Excavation inside the coffer-dam was started immediately, and after several hours' work the obstruction was removed, releasing the water, which in the meantime had submerged the surrounding locality. The bracing of the coffer-dam with 12x12-in. timbers was now started, and continued as the excavation was started down. To permit the entrance of water, an opening was left in the coffer-dam, and a screen of 12x12-in. timbers set vertically was built.

The uncertainty of the condition of the sewer below the coffer-dam, and the extreme danger of the opening at this point becoming closed in case of a heavy rain, made it advisable to sink another shaft at the manhole on 33d St., just above Liberty Ave., on the opposite side of the railroad, and from this shaft construct a flume leading to a point just above the coffer-dam. The shaft at the manhole is 20 ft. square. The flume is 10 ft. wide and 200 ft. long, with a grade of 1%; the average depth of cut for the flume was about 12 ft.

The flume serves the double purpose of a bypass around the break, to make possible and easy the cleaning out of the sewer above the break, and of providing an additional waterway for storm flow. It was practically completed on the night of Dec. 6. The rain on the night of Dec. 6 and the day following put it to actual test, and demonstrated the wisdom of the extra precaution taken.

While work was in progress in connection with opening up the sewer at the places previously mentioned, the railroad company was busy excavating a ditch along and outside of the tracks, in accordance with previous arrangements. The ditch confined the water, thereby keeping it off the tracks and diverting the flow from the line of the damaged sewer.

A decision having been reached as to the policy to be pursued with respect to permanent repairs (which con-

templated the performance of the work by competitive bidding based upon unit-prices), it was necessary, in order to determine the character and extent of such repairs, to uncover the sewer from the west end of the bottling house on Sassafras St. to Cayuga St. The work of uncovering the sewer between the above points will require (under the most favorable weather conditions) at least two months, and during this interval all drainage will flow in the open ditch. To prevent damage to the railroad property in case of rain during this period, it was considered advisable to enlarge this ditch to regular lines and to sheet and brace the sides. The ditch is 2800 ft. long and 10 ft. wide, with a minimum depth of 6 ft. The sides are sheeted with 2-in. plank, 12 ft. in length, and braced transversely with 6x8-in. timbers.

Additional funds were appropriated, amounting to $60,000, making a total appropriation of $110,000 to cover the cost of the emergency work. The uncovering of the broken sewer from the brewery along Sassafras St. to the end of the stable of the brewery requires the use of a cableway, supported by three towers. The excavated material is loaded into wagons and hauled to a near-by dump, as there is no room on either side of the sewer to store the material. This work, together with the enlargement and sheeting of the open channel, is now in progress. Beyond the stable, where space is available, a different method of excavating and disposal of material will be adopted.

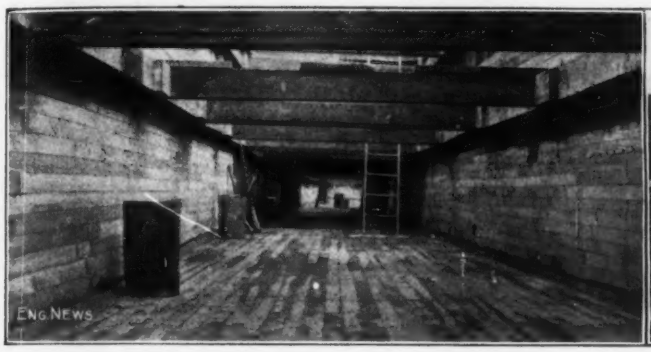

FIG. 10. FLUME DIVERSION IN YARDS NEAR FORFAR ST. BRIDGE

FIG. 11. SEWAGE FLOWING ALONG P. J. R.R. TRACKS, NOV. 26, 1913. LOOKING TOWARD 33D ST.

PLANS FOR REPAIRS AND FUTURE CONSTRUCTION

The formulation of a policy for the repair and reconstruction of the damaged sewer was necessarily held in abeyance until a more complete knowledge of the condition of the sewer was obtainable. The problem was complicated by the fact that the existing sewer was inadequate in capacity to provide proper drainage for the district served. The trunk sewer and main laterals of the system are surcharged during heavy rains, blowing off manholes, overflowing, and backing up the sewage into the laterals and connections. For the improvement of the drainage of the basin a study was made two years ago of the drainage requirements and designs were prepared for a relief sewer, which has since been completed, paralleling the existing main between Aiken and Winebiddle Aves. It was anticipated that this relief sewer would be extended to the river as funds became available. The capacity of the damaged sewer, as before stated, was 800 cu.ft. per sec., while the total required discharging capacity is 2000 cu.ft. per sec. This computed runoff is based on an area of about 1600 acres, the time of concentration of the flow with sewers full at the point computed being 31 minutes, the coefficient of runoff taken at 0.63 and the rainfall rate 2 in. per hr.; the estimated quantity provides for future development. In view of the foregoing, it was necessary to decide whether, under the circumstances, it would be advisable to permanently repair the old sewer to its former dimensions or to construct an entirely new sewer adequate to carry the total runoff of the watershed. In case the first plan was adopted, the construction of a relief sewer equivalent to a circular sewer 9 ft. in diameter would be required to provide adequate drainage. A new single sewer, adequate to carry the total runoff, would have a carrying capacity equivalent to an 11-ft. circular sewer.

The position of the State Commissioner of Health with regard to the continued use of the combined system and the discharge of the untreated effluent into the river further complicated the problem. At the request of the municipal authorities, the State Commissioner of Health, Dr. Samuel G. Dixon, was represented in the consideration of these matters by F. Herbert Snow, Chief Engineer of the State Department of Health. After examining the situation and conferring with the local engineers, a joint report was submitted at the request of Council, by Mr. Snow and Mr. Sprague. The report makes definite recommendations with regard to the needs for additional drainage facilities in the whole basin, the policy to be followed with respect to repairing the damaged sewer, and the matter of the disposal of the sewage into the Allegheny River.

Aside from relative economy of construction, the following factors entered into the determination of plans for rebuilding and reconstruction. It was necessary to provide at the earliest possible moment a temporary waterway for the sewage to avoid damage from flooding and to relieve unsanitary conditions. The construction of a new sewer 11 ft. in diameter, adequate to carry the entire discharge from the basin, would involve serious delay, pending the procurement of funds by a bond issue. Funds for repairing the sewer to its present size could be readily obtained by tax levy. During the construction of the 11-ft. sewer the flow would have to be maintained in an open channel. The time required to construct the larger sewer would greatly exceed that required to repair the present sewer. Since the invert of the sewer was found to be intact, it is feasible to satisfactorily re-

pair the sewer by constructing a new arch, the flow in the meantime being confined in the old invert. Repairing of the present sewer was further justified by the fact that when so repaired it will have a further life of at least 25 yr.

It was further determined that the cost of permanently repairing the existing sewer with a new arch and subsequently building a 9-ft. relief sewer was less than the cost of building a single new sewer, particularly since the increased cost of maintaining the open channel required by this latter plan would be considerable and the plan itself open to serious objection.

After considering the situation from all standpoints, definite recommendations were made, in the joint report referred to, to the effect that the old sewer be repaired to its original size, at an estimated cost of $250,000, and later, under funds to be provided by bond issue, a relief sewer be provided, the total cost of which work, inc'uding additional relief sewers for the main laterals, is estimated at $938,000.

The accompanying general plan shows the existing combined sewer system of the basin, relief sewers recently completed or now under construction, and proposed relief sewers, which, with the existing sewers, will provide adequate drainage for the whole watershed.

The question of the treatment of the sewage from the basin was deferred until such time as detailed plans for the comprehensive improvement of the drainage facilities of the basin can be prepared and funds provided for their execution. The probable solution of the problem will consist of a dry-weather diversion and the installation of settling or screening devices, or both, with a submerged outlet extending to the thread of the Allegheny River. The construction of such works will probably be delayed until required by the State Department of Health.

The work is under the supervision of N. S. Sprague, Superintendent of the Bureau of Construction; I. Chas. Palmer is Division Engineer. Ottomar Stange, of the firm of John F. Casey Co., has supervision over the contractor's forces, assisted by G. M. McAllister, Superintendent.

Water-Works Improvements at La Crosse, Wis.

SYNOPSIS—Modern demands for a better quality of water, better fire protection and improved pumping conditions are leading to an abandonment of the Mississippi River and direct pumping for a supply pumped by electric power from wells in the gravel and sand to a receiving reservoir, from which the water is pumped to a distributing reservoir. The old pumps are being overhauled and moved to a new station near the receiving reservoirs. The well pump houses, the new main pumping station and the two reinforced-concrete covered reservoirs are described.

The city of La Crosse, Wis., is located on the east side of the Mississippi River and is divided into two parts by the La Crosse River, which enters the former stream at this point. The population was about 30,500 in 1910, and is estimated to reach about 40,000 by 1930. The present water-supply system was built by the city in 1877, taking water from the Mississippi River and pumping it directly into the mains. The water is not filtered.

The consumption has averaged nearly 3,000,000 gal. per 24 hr. for about ten years. The most severe conditions were during the very dry summer of 1910, the daily average for July being 5,500,000 gal., with a maximum of 6,800,000 gal. for a few days.

Old Water-Works System

The pumping station was located near the Mississippi River, in the manufacturing district of the southern part of the city, closely adjacent to the business center. With such a location it had a rather severe fire hazard.

A change in the source and mode of supply became desirable to secure better water and to avoid the difficulties and limitations incident to a direct-pumping system. Accordingly, Alvord & Burdick, consulting engineers, of Chicago, were engaged in 1911 to make investigations for a new supply and for other improvements in the works. They advised that water sufficient for the next 15 or 20 years could be obtained from wells sunk in a 240-acre tract of sand and gravel land in the La Crosse Valley, within the city limits, and that this supply could be enlarged by sinking more wells in adjacent land when needed. They recommended that the water be pumped from the wells by electric power, delivered to a 1,000,000-gal. covered storage reservoir and pumped thence to a 5,000,000-gal. reservoir, from which the city would be supplied by gravity. The plan included moving the old pumping plant to a new station near the receiving reservoir and making some improvements in the largest pump.*

These recommendations were adopted, and the works are now well advanced. As soon as the works are completed, the river supply will be abandoned. The contracts were let in November, 1912, and the entire system is expected to be completed by January, 1914. The construction has been under the direction of Alvord & Burdick, with L. A. Genpel as resident engineer in charge. The city engineer is Geo. P. Braddish. The estimate of cost for the new water-supply system was as follows:

Land (240 acres)	$16,300
Wells, 19 at $650	12,350
Well curbs, houses and connections	11,000
Motors, pumps and generators	25,000
Pipe lines to wells	14,260
Transmission lines	2,400
Reservoir, 1,000,000 gal.	15,000
Pumping station	45,000
Pumping engine, 8,000,000 gal.	28,000
Piping, moving old engines and boilers, etc.	32,000
High-service reservoir, 5,000,000 gal.	70,000
Distribution mains	95,610
Total	$366,920
Contingency and incidental 15%	55,000
Grand total	$421,920

*The principal machinery at the old station was as follows: One 10,000,000-gal. Holly vertical triple expansion engine (1895), one 4,000,000-gal. Blake compound condensing direct-acting engine (1884), one 2,500,00-gal. engine of this latter type (1881), three 187-hp. internally fired boilers, 8x16 ft. (1902). The total lift at the old plant was about 168 ft.

The Wells

For the new water-supply there are five groups of four wells each. The wells are 115 to 160 ft. deep, 10 in. diameter, with a 20-ft. length of strainer on the bottom. The suction pipe is connected directly to the well casing, no interior drop-pipe being used. In each group of wells, a 10-in. flanged suction pipe runs from each well to the low-service pumping station of the group; and 14- and 20-in. discharge pipes leading from these five stations unite in a 24-in. main terminating at the low-service reservoir.

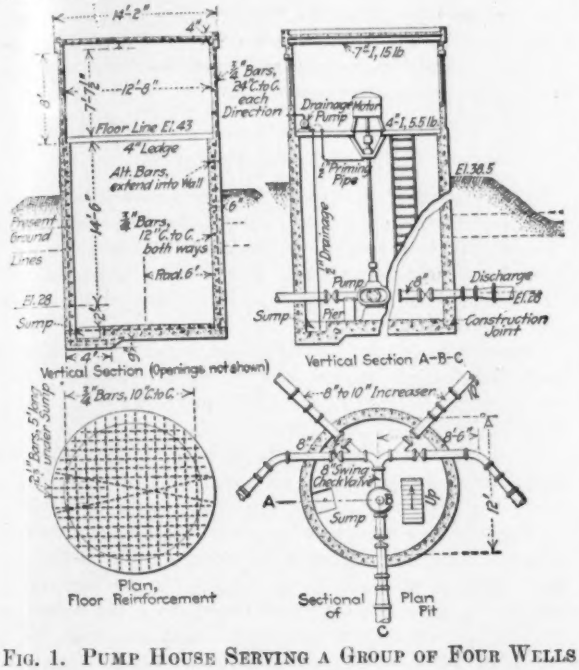

Fig. 1. Pump House Serving a Group of Four Wells

The wells in each group are about 100 ft. apart, and the groups are 800 to 900 ft. apart, being so arranged that wells for additional groups can be put down to increase the supply by 30 to 50% on the same site.

The wells were put down by an ordinary well-drilling rig. Difficulty was anticipated in laying the pipe lines across the marshy ground but the work proved to be comparatively easy. Steel sheet piling was used for these pipe trenches, being driven to below the grade line, and removed after the pipes were laid. This work was done during the winter.

Electrically Driven Pumps at Wells

Each group of four wells is served by an electrically driven pump in a pump house central to the group. The pump is of the vertical-shaft centrifugal type, having a capacity of 1400 gal. per min. (2,000,000 gal. per day) at 1120 r.p.m. against a head of 40 ft., including suction. On the top of the shaft is mounted a 25-hp. 440-volt, 60-cycle, three-phase motor. Ordinarily four of the groups will be pumped and one held in reserve. Current is brought to the plant from the main pumping station, the poles carrying the transmission line also serving to carry telephone wires.

The pump houses, Fig. 1, are circular structures of reinforced concrete, 12 ft. 8 in. diameter inside, with a height of 16 ft. 6 in. from the bottom of the pit to the pump floor, and 8 ft. 9 in. from this floor to the roof. The bottom is at elev. 26 (above city datum). The walls are 12 in. thick in the lower portion and 6 in. above, while the floor is 15 in. thick and the roof 4 in. The pump floor is of steel plates on I-beam framing, and the roof has an I-beam for a hoist in handling the machinery. The concrete is similar to that used elsewhere.

A small electrically driven drainage pump is provided in the pump house and serves also for priming the main pump. To provide against the possibility of delivering drainage water from the sump into the mains, a flexible hose connection is used on the small pump so that it can be connected either to the main pump or to the sump.

Construction—The well houses extend about 8 ft. below the ground-water line, so that special means had to be employed in building them. The contractor drove sheeting to a depth of about 11 ft., forming an octagon of about 17 ft. diameter. The ground was then excavated and a wooden tub or tank set in place to form the outside of the form, the concrete floor being laid on the bottom. When the concrete had set, the tank was taken down for use at another pit, but of course required a new wood floor each time. As the marsh is overflowed every year, the walls had to be made watertight. The drawings show the earth banked up around the pump houses, but part of this banking was washed away by the spring floods, necessitating the use of a ladder with platform landing instead of the stairs originally planned.

Receiving Reservoir

The 24-in. discharge main from the wells delivers the water into a receiving reservoir of 1,000,000-gal. capacity at the new pumping station in Myrick Park. A Venturi meter is placed in this main, and has a throat diameter to measure water accurately at rates varying from 1,400,000 to 13,000,000 gal. daily. The reservoir is rectangular in plan, and is divided into two chambers. The main or storage chamber is 102x97 ft., and the suction chamber is 9x102 ft. The depth is 14 ft. The reservoir is below the ground level of the park, and its roof is covered with about 12 in. of earth.

The reservoir is of reinforced-concrete construction, somewhat similar to that employed for the distributing or service reservoir. The columns are spaced 12 ft. 1 in.

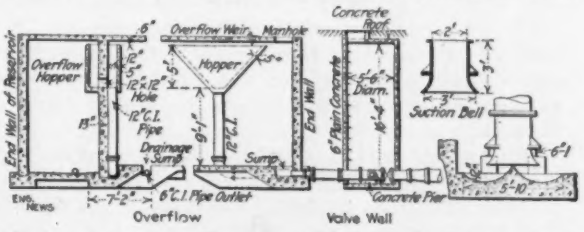

Fig. 2. Overflow Trough and Suction Bell of the Receiving Reservoir

longitudinally, and 13 ft. 6 in. and 14 ft. 7 in. transversely. Upon them are the 23x12-in. girders with a 6-in. roof slab. The concrete floor has a ¾-in. finishing coat of 1:2 cement mortar. A ventilating shaft is provided and to prevent the pocketing of air between the girders, 4-in. vent holes are provided in these just below the slab.

The overflow device is a concrete trough, with pipe connections, as shown in Fig. 2. To insure a uniform

draft, the suction pipe of each engine ends in a bell directly over an annular depression in the floor of the suction chamber. This arrangement for the Blake pump is shown in Fig. 2.

Main Pumping Station

This new pumping station is located in Myrick Park, close beside the receiving reservoir at elev. 50. It pumps the water through a 24-in. main to the distributing reservoir (elev. 258), a distance of 1½ miles, and has also a 30-in. main connected to the city distribution system for direct fire service. The 24-in. and 20-in. distribution mains are connected also with the distributing reservoir by a 20-in. main. Being located in a public park, the building has been designed with a view to a pleasing appearance. It is a one-story building with exterior walls of red brick and white terra cotta, while the interior is of buff repressed brick with white marble wainscot. The roof is of concrete, on steel trusses, the slab being covered with an asphalt composition and gravel. The coal bunker is of reinforced concrete.

The concrete foundation walls are exposed for about 3 ft. above the ground level and this part is given a special treatment. The concrete is wetted and given a coat

Fig. 3. Grand-dad Bluff, La Crosse, Wis.
(The location of the high-service reservoir is indicated by the tower and chute of the concreting plant.)

of 1:3 cement mortar. This also is wetted and then given a finishing coat of mortar composed of 1 part portland cement and 4 parts crushed white granite screenings (with no sand). When the latter coat has set, it is scrubbed with acid to remove the surface cement and expose the granite particles, the treated surface being well washed down with a hose to remove all traces of the acid.

The engine room is 100x50 ft., with its floor 8 ft. above that of the boiler room, 57x40 ft. The coal bunker and shop (alongside the boiler room) are 57x29 ft. The bunker has a storage capacity of 500 tons, and is served by a spur track from the adjacent line of the Green Bay & Western R.R. The smokestack is of concrete, 150 ft. high, with inside diameter of 5 ft. The engine room is served by a 15-ton overhead electric traveling crane of 50-ft. span, with a headway of 32 ft.

In the engine room are being installed the old Holly engine (with plungers reduced to give 8,000,000-gal. capacity at an ordinary pressure of 90 to 100 lb.), and a new horizontal cross-compound high-duty pumping engine of the same capacity, against a total head of 240 ft. (including suction). Space is provided for a third engine in the future, but at present the old Blake 4,000,000-gal. direct-acting pump will be installed here for emergency purposes.

In this room also are the turbo-generator units for supplying current to the low-service pumps at the wells. These are of 125 kv.a., and two are now installed, while space is provided for a third.

The boiler room provides for four internally fired boilers, of 187-hp., and the three of these from the old station are now being transferred to the new station.

The concrete foundations for the pumping station and the engines were built during the winter, in order to be in readiness for installing the pumping engines early in the spring. The materials were heated, and the fresh concrete carefully covered and protected, and in only a few spots was the concrete found to be at all open or granular.

Distributing Reservoir

The distributing or service reservoir is located on a bench or shelf on the north side of the Grand-dad Bluff, which rises to a height of about 600 ft. above the river. Fig. 3 is a view of the bluff. The location of the reservoir is indicated by the tower and inclined chute of the concreting plant.

The reservoir is of 5,000,000-gal. capacity, 20 ft. deep, and its bottom is at elev. 258, giving a pressure of 77 to 90 lb. in the city district and 84 to 96 lb. at the pumping station, according to whether the reservoir is full or empty. A 20-in. main leads to a connection with the distribution system in the city, and a 24-in. main is laid from the new station to a connection with this system.

The reservoir is of peculiar shape in plan, as shown by Fig. 4. This is in order to conform closely to the contour lines of the hillside and keep the bottom on the original solid ground, thus reducing to a minimum the amount of excavation required. The reservoir has a width of 60 to 80 ft., and a total length on the center line of about 460 ft., while the area is about 33,000 sq. ft. It is divided into two parts of approximately equal size by means of a partition wall.

The reservoir is built on a foundation of compact clay, with large boulders, the character of the clay being such that the excavation would stand with an almost vertical face. It is banked and covered with earth, as shown in Fig. 7, the normal depth of cover being 1 ft., but along the inner side this may be increased by material sliding from above. Around the upper side is a stone-paved gutter, to intercept water flowing down the hillside.

The side walls, Fig. 5, are 20 in. thick to a height of 12 ft. and then batter on the inside to a top width of 11 in. They are reinforced as shown, the reinforcement being designed for full water pressure on the inside and for earth pressure equivalent to half water on the outside. The partition wall batters on both sides and is designed, of course, for full water pressure on both sides. The walls were built in sections and for making the vertical joints, a stick 4x6 in. was set upright in the end of the form, placed diagonally to the wall and having a sheet-iron plate let into its projecting corner. This part of the plate was embedded in the concrete, and when the forms were removed, the 4x6-in. stick was taken off, leaving the other half of the plate to be embedded in the concrete of the adjacent section.

The concrete floor is reinforced throughout, with extra reinforcement at the column footings. It is formed with

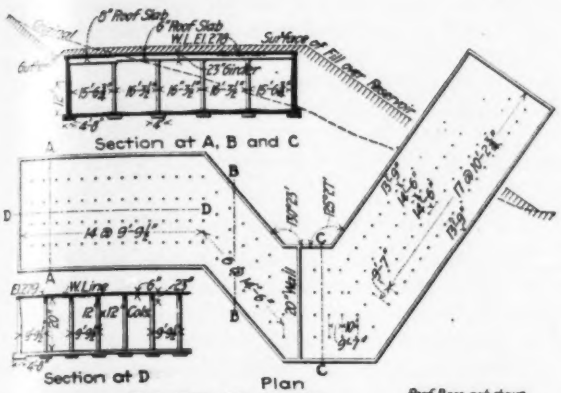

FIG. 4 PLAN AND SECTIONS

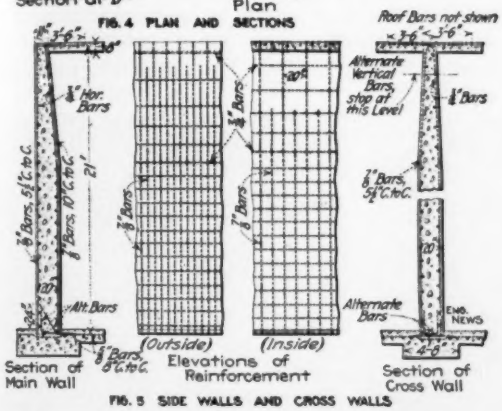

FIG. 5 SIDE WALLS AND CROSS WALLS

Each of the two compartments of the reservoir has two ventilator manholes, which are rectangular chambers extending above the roof as shown in Fig. 7. The chamber is 2 ft. square inside and 3 ft. high, with a hinged iron cover on the top and an opening 12x24 in. in one side, fitted with louvers. Each compartment has a 12-in. overflow pipe with 24-in. funnel, the pipes passing through the walls and uniting in a single 12-in. pipe discharging in a cobble-lined surface gutter (Fig. 10). In the floor of each compartment also is a 20-in. inlet pipe and 12-in. blowoff pipe, the pipes being fitted with cast-iron sleeves which have flanges embedded in the concrete (Fig. 10).

CONSTRUCTION METHODS—The excavation for the high-service reservoir was done with an orange-peel bucket handled by a locomotive crane. The concrete was mixed in a drum discharging into an elevator bucket traveling in a tower 160 ft. high (Fig. 3). From a hopper near the top of the tower the concrete was delivered to a galvanized-iron inclined chute of semi-circular section. This ended in a receiving hopper from which the concrete was wheeled to the forms in buggies. The chute was shifted as required, so as to keep the hopper near the work of concreting. No concrete was spouted into place in the forms, the only concrete poured directly from the chute being at a low place beneath the floor level which was filled with about 700 cu.yd. of concrete (1:3½:7).

REINFORCED CONCRETE

All foundation concrete throughout the work is a 1:3:6 mixture (using ⅛ in. to 1½ in. broken stone).

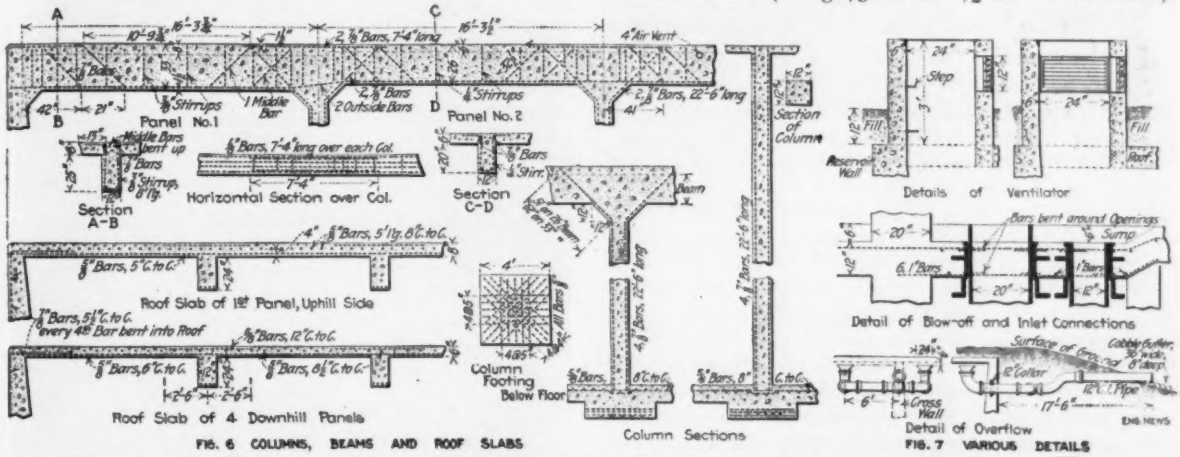

FIG. 6 COLUMNS, BEAMS AND ROOF SLABS

FIG. 7 VARIOUS DETAILS

FIGS. 4 TO 7. PLAN, SECTIONS AND DETAILS OF THE DISTRIBUTING RESERVOIR AT LA CROSSE, WIS.

longitudinal gutters between the rows of columns, the gutters having a slope of about 1 in 500 toward the blowoff. The floor has a ¾-in. finish coat of cement mortar 1:2. There are four rows of columns, spaced transversely according to the varying widths of the reservoir, and with a longitudinal spacing of 9 ft. 9½ to 14 ft. 6 in.

The columns carry transverse girders, the section and reinforcement of which vary with the different lengths of span, as shown in Fig. 6. The roof slab is mainly 6 in. thick, designed for 1 ft. of earth cover. In the panel next the hillside, however, it is made 8 in. thick, to provide against an additional load due to material sliding upon it from above and increasing the depth of cover to 4 ft.

For concrete which is to be reinforced, the proportions are 1:2:4 (with stone as above), and where it must be waterproof there is added to the cement 10% (by volume) of hydrated lime. The reinforcement is designed for a stress of 16,000 lb., and consists of high-carbon plain or deformed bars of 50,000 to 60,000 lb. elastic limit, all rolled from billets. Splices are avoided as far as possible, but where necessary the ends of the bars are lapped for a distance not less than 50 times their diameter. In the roof slabs, no bent bars are used, but the bars in the lower layer are laid through the column heads, while short pieces of straight bars are laid near the upper surface of the slab, over the columns (Fig. 6). This requires a little extra steel, but is much more convenient in construction.

A New Type of Reinforced-Concrete Floor Construction

A reinforced-concrete floor system of a ribbed-slab type, in which the spaces between the ribs are formed by steel troughs, constitutes one of the special structural features of the new Railway Exchange Building, at St. Louis. This construction combines lightness with strength, and effected a material reduction in weight as compared with the floor system originally designed.

The Railway Exchange is an unusually large store and office building, covering an entire city block between 6th, 7th, Locust and Olive Sts. It is 228x271 ft. in plan, with 21 stories, giving a height of 308 ft. It covers a ground area of 62,000 sq.ft., and has a total floor area of over 30 acres. The foundations are concrete piers

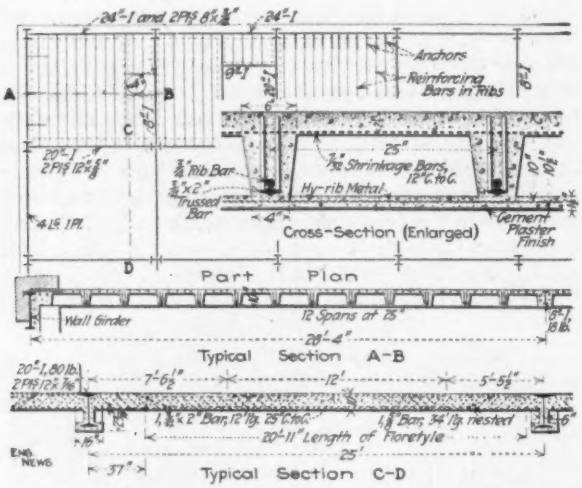

FIG. 1. STEEL FLOOR FRAMING AND RIBBED-CONCRETE FLOOR CONSTRUCTION IN THE RAILWAY EXCHANGE BUILDING, ST. LOUIS, MO.

carried down to rock. The columns and girders are of steel, with concrete floors and roof and terra cotta exterior finish. The architects were Mauran, Russell & Crowell, of St. Louis, and the structural design was prepared by the Westlake Construction Co., of St. Louis, which company had the general contract. The ribbed-slab floor construction with steel "floretyles" was designed by the Trussed Concrete Steel Co., of Detroit, Mich.

The steel columns are of H-section and the girders are I-beams. In some of the panels on the lower floors (with the heavier floor load) the beams have top and bottom flange or cover plates. The assumed loading for the floors was 135 lb. per sq.ft. for the seven store floors and 95 lb. for the 14 office floors. This is the total superimposed load, including finish and live-load. The typical floor panels are 26½x25 ft. c. to c. of columns, but the end panels are 28x25 ft. at one end and 27½x25 ft. at the other end. Some of the panels are subdivided by framing for elevators and stairways. The ceilings are flat, being formed by a thin slab at the bottom of the floor ribs.

The concrete-slab construction of the floors as originally designed gave a weight of about 73 lb. per sq.ft. The advantages of the ribbed design were presented, however, and as these were substantiated by a loading test of a full-sized panel this construction was adopted. Its weight is a little under 50 lb. per sq.ft., or a saving of nearly 33% in dead load as compared with the original design. This results in a considerable reduction of the load to be carried by the floor framing, columns and foundations. Fig. 1 shows a typical floor plan and cross-sections in both directions of the panel, and gives also an enlarged cross-section showing the details of the reinforcement.

"FLORETYLE" FLOOR CONSTRUCTION

The special feature, as noted already, is the use of steel troughs for the spaces between the ribs. These are given the trade name of "floretyles," and are of thin sheet steel deeply corrugated, made in lengths of 3 to 4 ft., and lapped 1 in. at the joints. In this building the "floretyles" are 7½ in. high, 19 in. wide at the top and 21 in. at the bottom. They are spaced 25 in. c. to c., so as to form tapering ribs 4 in. wide at the bottom. The thickness of concrete over the troughs is 2½ in., giving a total effective depth of 10 in. for the ribs. In the longer spans already mentioned this is increased to 10½ in. Fig. 2 shows the "floretyles" in place, with the reinforcing bars (for the ribs) between them. It shows also (at the top) the very simple centering required.

In building the floor, a layer of three-rib Hy-rib metal for the flat ceiling was first laid upon the centering of each panel, and upon this (at right angles to the metal ribs) were placed the rows of steel troughs, their position in relation to the girders being as shown in Fig. 1. In each open space were laid two Kahn bars of medium openhearth steel, one of these a trussed bar ½x2¼ in. and the other a ¾-in. rib bar. The combined area of the bars was 2.2 sq.in. The ¾-in. bars were bent up over the I-beam girders and lapped with those of the adjacent span, giving 1.12 sq.in. of steel over the girders at 25-in. intervals. On top of the "floretyles" and at right angles to the main reinforcement were laid $\frac{7}{32}$-in. shrinkage bars 12 in. apart.

The roof is of similar construction, the concrete being covered with the Johns-Manville Co.'s built-up asbestos roofing. This is composed of layers of asbestos felt (placed separately), cemented together and coated with Trinidad asphalt.

The "floretyles" have deep ribs across the top and heavy corners to insure stiffness in supporting the loads and resisting deformation. Their notched edges engage with the meshes of the Hy-rib ceiling sheet and assist in holding these parts together. The troughs are normally 20 in. wide at the base and 6 to 12 in. deep, made in lengths of 3 ft. and 4 ft. Owing to their taper form they are easily nested for shipment, and they are not liable to breakage in transportation. In concreting also, there is no loss of cement or concrete through flow at joints or broken spots. In addition to the Railway Exchange Building at St. Louis, this construction has been employed in several buildings, including offices, hotels and schools; also for the Capitol building in Formosa, for the Japanese government; the president's palace at Havana, for the Cuban government; and the Washington Hotel at Colon, for the United States government.

TEST OF FLOOR PANEL

In the loading test (already noted) of the floor as designed for the Railway Exchange Building, the test panel represented an end or wall panel of a continuous

floor, and in order to approximate the conditions existing in such a floor the beams and floor were extended as cantilevers on three sides, the extension being 7 ft. 6 in. beyond the panel line on two sides and 7 ft. 2 in. on the other side, as shown in Fig. 3. These cantilever portions were loaded equally with the panel. The size of the panel proper was 25x26½ ft. c. to c. of the columns, which were of reinforced concrete 24 in. square. In the longer direction, the panel was supported by two 24-in. I-beams on each side, and these were connected by single 8-in. I-beams on the shorter sides. The beams were embedded in concrete fireproofing.

The floor was designed to carry a live-load of 150 lb. per sq.ft. The steel troughs and reinforcement were laid as described above, and then the concrete (1 : 1½ : 3) was poured to a thickness of 10 in. from the bottom of the Hy-rib ceiling, giving 2½ in. of concrete above the "floretyles" and providing the necessary toe-section required in compression. The Hy-rib metal was given a ¾-in. coat of cement mortar (1 : 3).

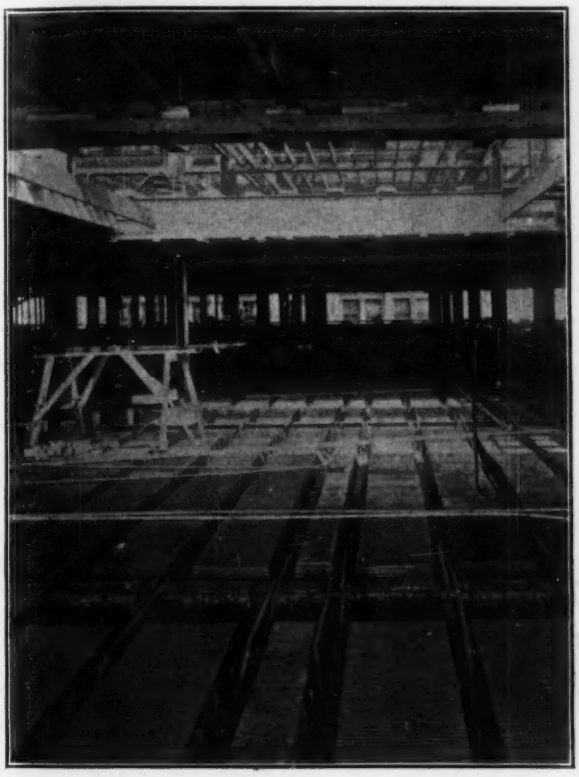

FIG. 2. STEEL TROUGHS OR "FLORETYLES" FOR RIBBED-FLOOR CONSTRUCTION, WITH REINFORCING BARS IN PLACE FOR THE RIBS; RAILWAY EXCHANGE BUILDING, ST. LOUIS, MO.

The concrete was poured Dec. 28, 29, 1911, during cold weather, and all materials were heated before using. As the panel was in the open, a tent was erected over it and fires were kept burning in the tent until the time for applying the test loads. The tent was erected immediately after the concrete had been poured, but owing to severe weather conditions the temperature of the slab dropped at once below freezing, and was not raised above 40° until ten days later, the slab being apparently frozen for four days. During the latter portion of the aging period the temperature in the tent was 55 to 70° F. The tent was removed two days in advance of the test.

The test load was placed Jan. 30, 1912. This was 33 days after pouring the concrete, but the slab had been exposed to a temperature of over 40° F. for only 21 days. Wooden sides were built around the slab, and loose sand was filled in during one day, the total amount being 491,081 lb. (on slab and cantilever portions) an average of 360.2 lb. per sq.ft. The load remained for about 35 hours. It was then taken off gradually, beginning Feb. 2, so that zero readings were not obtained until Feb. 6, 1912, and final readings were made on Feb. 8.

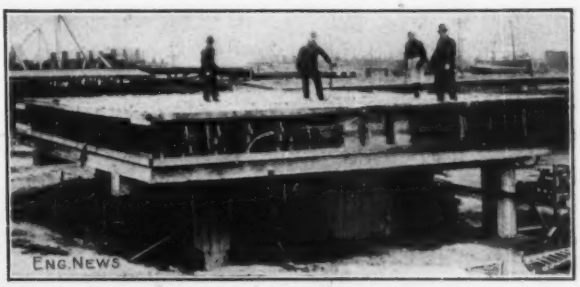

FIG. 3. LOADING TEST OF A RIBBED CONCRETE FLOOR PANEL BUILT WITH STEEL "FLORETYLES"

The weight of the floor panel was about 50 lb. per sq. ft. The test required by the city ordinances is a superimposed load equal to the dead load plus twice the live load, which in this case would be 350 lb. per sq.ft. The center point of the panel deflection was less than 0.5 in. for a 360-lb. load, which is well within the limits provided for in the ordinance. The permanent set was about 0.2 in. This test was carried out by Robert W. Hunt & Co., of Chicago, and the above particulars are taken from their report.

"FLOORDOME" FLOOR CONSTRUCTION

A modification of the floor system above described consists in the use of rectangular bowls or "floredomes" of corrugated steel, instead of the continuous troughs or "floretyles," thus forming a cellular floor with ribs and steel reinforcement in both directions. A flat ceiling is formed in the manner already described. This also is a special design by the Trussed Concrete Steel Co.

The "floredomes" are 21½ in. square at the base, and 6 to 12 in. deep. They are nested for shipment, thus facilitating transportation materially. The Mt. Tabor school, at Portland, Ore., and the Packard Service Building at Los Angeles, Calif., are among the buildings in which this floor construction has been adopted.

Engineering vs. Bricklaying—The following item appeared in a New York newspaper on Dec. 20:

Because of the complaints of the Bricklayers' Union the Public Service Commission yesterday ordered that all waterproofers employed on the new subway work should receive $5.60 a day instead of $3.25, and that after Jan. 1 they should receive $6. The union represented to the commission that as the waterproofers were working in mastic, their work really was that of bricklayers, and that therefore they should receive the pay of bricklayers.

This is greater compensation than is paid by the Commission to a considerable number of engineering graduates of some years' experience employed by the Public Service Commission itself.

Report on Collapse of Steel Frame of a Theater; Cause a Mystery

On Jan. 22, 1913, a large new theater in New York City, under construction and nearly completed, was wrecked by the collapse of most of its roof and the interior balcony. Little more than the walls and auditorium floor remained. The collapse occurred only 20 minutes after quitting time and the last of the 200 workmen employed on the building had only just left when the structure fell. Fig. 1 shows the interior just after the collapse.

A brief account of the accident was published in our issue of Jan. 30, 1913, p. 234, but at that time the steel framing was largely covered by the débris and it was impossible to report on the condition of the parts and the circumstances as shown thereby. Later examination showed a remarkable and puzzling state of affairs. We have refrained from dealing with it until the issuance of the report on the collapse by the Bureau of Buildings of Manhattan Borough. This report, rendered last May, by Alfred Ludwig, chief inspector, to R. P. Miller, Superintendent of the Bureau, was not made public until a few weeks ago.

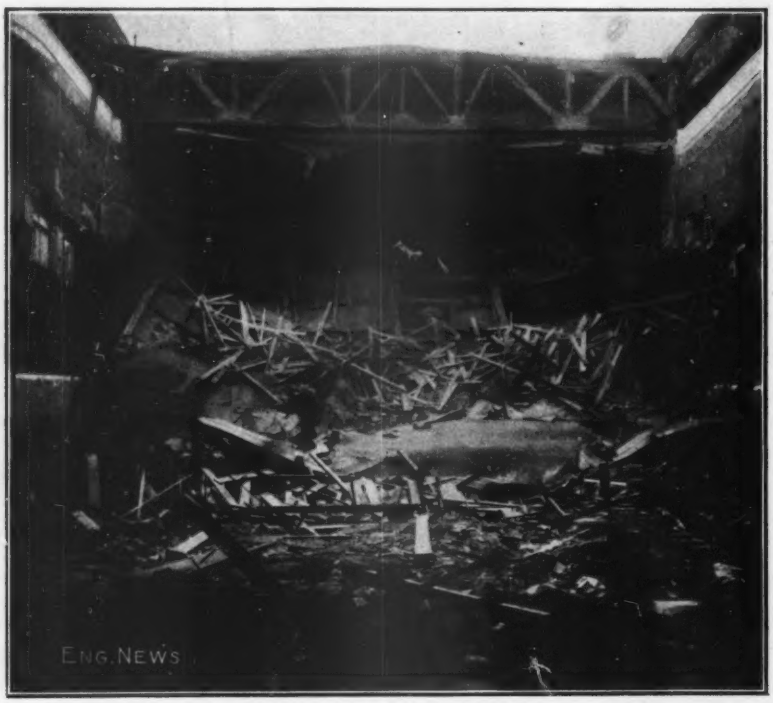

FIG. 1. INTERIOR OF ORPHEUM THEATER, NEW YORK, ON DAY AFTER COLLAPSE

Other information is also available now: We have secured an opinion from the steel contractors, National Bridge Works, by T. J. Foster, President; and have copies of reports made by four consulting engineers retained by the general contractor, Fleischmann Bros. Co., to investigate the collapse: Robt. W. Boyd, Laurence A. Ball, Thos. J. Bird, and Weiskopf & Burroughs, all of New York City. The four investigations were made independently of each other and of the investigations by the Bureau of Buildings and by the steel contractor.

The various opinions on the collapse differ radically. The engineers outside of the Bureau of Buildings present *three different explanations* of cause. Chief Inspector Ludwig's report, on the other hand, considers all three explanations as open to objection, and does not decide in favor of any. A further remarkable feature of the case is that each of the three explanations is highly interesting in itself, two of them at least involving conditions practically unprecedented in structural failures.

The points involved in the collapse include: tension failure; buckling of columns; stability of a cantilever system of support; overloading by excess concrete; excessive stresses; and quality of material.

THE BUILDING

The Orpheum Theater, 168-180 East 87th St., New York City, was being built by Fleischman Bros. Co. for the Loew Theatrical Enterprises. Thos. W. Lamb was architect for the latter. The National Bridge Works, New York City, built and erected the steelwork. The steel erection was begun in the early part of November, 1912, and the last of the truss work was set before the end of December. Floor and roof concrete, plastering, etc., went on in January. On the day of the collapse, the interior trim and some minor framing alone remained to be placed.

The structural part of the building is drawn in Fig. 2, containing plan of roof, plan of balcony, and longitudinal section. In the two plans the portion which collapsed is indicated.

The roof over the auditorium was carried by two large transverse trusses, the rear one resting on the side walls and the forward one resting on two columns, each about 7 ft. inward of the walls. The two trusses supported longitudinal secondary trusses and transverse I-beams.

The balcony was of the usual cantilever construction. A transverse truss resting on two short columns near the side walls carried the balcony. Three steeply inclined cantilever trusses extended over this transverse truss, their rear ends being attached to the floor framing in the rear part of the theater at one of the upper levels, while the forward end carried the plate-girder balcony fascia rail. The balcony fascia girder, curving forward along the sides, extended as far as the high columns supporting the forward roof truss, and the ends of the girder were attached to these columns. Some beam framing for boxes at the balcony level between this column and the proscenium wall was not in place at the time of the collapse. With this exception the balcony framing was complete, and, as already mentioned, the concrete floor slab, stepping, etc., was in place.

The auditorium floor was directly on the ground.

THE COLLAPSE

On Jan. 22, 1913, the workmen in the building left at 5 o'clock; about 5:20 the roof and balcony fell in.

There was no extra weight on roof or balcony, and everything was quiet a moment prior to the collapse. No indications have been found that point to any kind of violence or malicious damage; the building apparently just broke down. No one was injured.

The plan sketch, Fig. 3, indicates the lay of the prin-

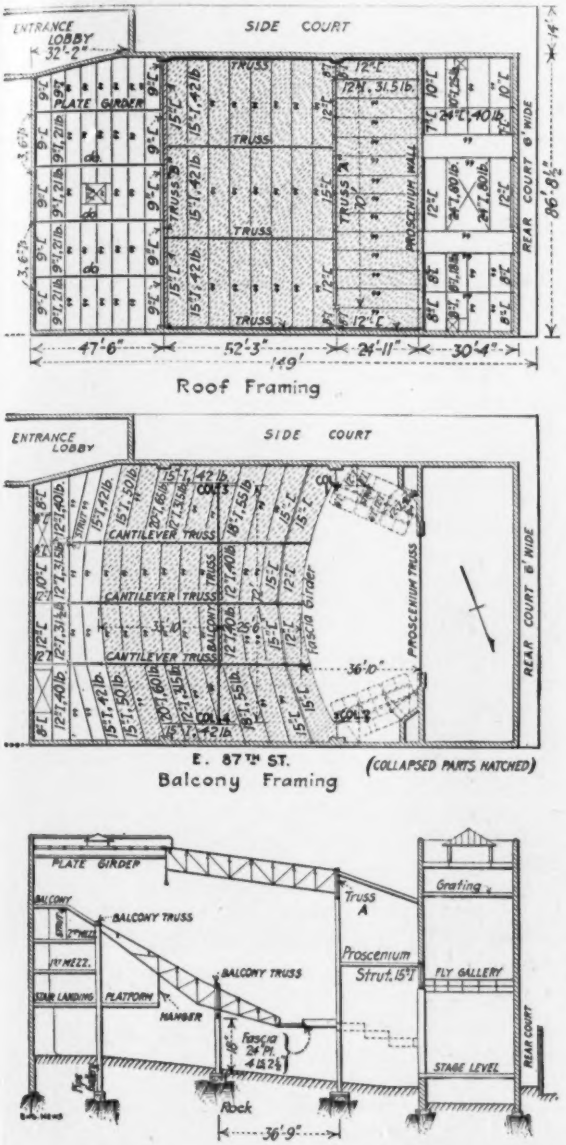

FIG. 2. STRUCTURAL STEELWORK OF ORPHEUM THEATER

cipal parts of the fallen framework after the accident, as determined when the concrete, timber and other miscellaneous wreckage had been removed.

All parts of the roof depending on the forward roof truss (the truss supported on the two long columns) fell; as indicated in Fig. 3, the truss moved back or eastward in the fall, no doubt because the longitudinal trusses pivoted around their attachment to the rear truss until they broke away, and thus pulled the forward truss back. The longitudinal trusses, as well as the forward truss itself, remained whole (without breaks) and still connected to each other at their junctions. The rear truss was somewhat damaged by the wrenching away of the longitudinal trusses, but otherwise stayed in place undisturbed. The two long columns supporting the forward roof truss buckled each in four segments, lying, as indicated in Fig. 3 and shown by Figs. 9 and 10, in the shape of a closely compressed and twisted Z. It was evident that *the roof fell because the long columns buckled under it.*

The entire balcony fell, with *buckling of its two supporting columns* and *breaking of the main transverse truss in the bottom chord.* The structure swung back (east) as the three cantilever trusses remained attached to the upper floor framing at their anchorage end. The south half of the balcony truss came down in practically vertical position, the center and south cantilevers breaking apart through the gusset and flange splice at the angle of the bottom chord. The north cantilever did not break, and tipped the north end of the supporting balcony truss (see Fig. 8) somewhat forward. The breaks in the two southerly cantilevers, being in the lower (compression) chord, doubtless happened when the front ends struck the ground.

The balcony fascia girder, 24 in. deep, remained attached to the two roof columns as well as to the three cantilever trusses. It was folded back on the balcony cantilevers in the middle portion.

PRINCIPAL BREAKS

The facts concerning the three important points of failure in the collapsed steelwork are noted as follows:

BALCONY TRUSS—Bottom chord broken squarely in two on the center line of the field splice, located between middle and north cantilevers. Just above here, top chord bent and partly broken, and several web members bent or broken (Figs. 6 and 7). Vertical of web showed well contracted tension failure, with reverse bends at top and bottom. Truss remained connected to its supporting columns at the end, except that the bottom-chord attachment at the south column was wrenched off, the column buckling between the two chords as well as below the truss. Truss chords of T-section (two angles, stem-plate and flange-plate). Bottom-chord splice which broke comprised side splice-plates and flange splice-plates. Break was clean and square, through rivet holes nearest the joint, the fracture faces of crystalline texture. Spliced chord-section was 2 angles 8x8x¾ in., 1 web 18x¾ in., 1 flange-plate 18x⅜ in.

BALCONY-TRUSS COLUMNS—Length 18 ft. from base to bottom chord of truss, 27 ft. from base to top chord. Section H-shaped, 12x12 in. (four angles and web plate). Cap plate of column 6 ft. above bottom chord of truss. and upper joint of truss developed into a shoe with flat base seated on the column cap and riveted to it (Fig. 12). Both columns broke or buckled at the bottom chord of the truss, and the south column also buckled at its cap (see views Figs. 13 and 14). The base of the south column moved off its grillage, but remained just alongside; the north column slipped far away from its grillage. The columns were unbraced, except by their connection to the balcony floor-slab, and at the top by a strut to the wall.

ROOF COLUMNS—Section H-shaped (four angles and web-plate), 12x12 in.; length 60 ft. from base to bottom chord of roof truss. Cap, and connection to truss, similar to detailing of balcony columns. Each column was held at top and bottom and at two intermediate

points. The points of bending or buckling *in both columns* are at the two points of bracing (the lower one where the balcony fascia girder was attached and the upper one where a 15-in. I-beam strut to the proscenium girder was attached), and at the bottom chord of the roof truss. The location and character of the bracing is shown by Fig. 4.

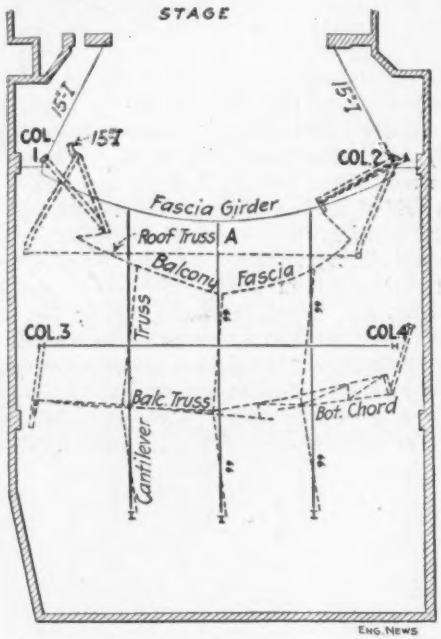

FIG. 3. POSITION OF MAIN STRUCTURAL PARTS AFTER COLLAPSE
(Sketched by R. W. Boyd.)

The tops of the two columns remained attached to the roof truss, and the bases remained close to their grillages, but tore away from them, without, however, destroying the footings. In addition, the columns showed some twist around the longitudinal axis. The general lay of these columns is shown in Fig. 3. Figs. 9, 10 and 11 are views of these columns.

CONDITIONS EXISTING BEFORE COLLAPSE

OVERLOADING BY EXCESS CONCRETE—The New York Building Code allows a steel stress of 16,000 lb. per sq.in. The transverse truss of the balcony at the time of the collapse was stressed in many of its main members to from 22,000 to 24,000 lb. per sq.in., and this with no live-load on the floor. The bottom-chord splice of the balcony truss was stressed to 22,000 lb. per sq.in. The cantilever trusses were similarly overstressed.

The balcony columns also were overloaded. A careful estimate of the actual weight in place, by Asst. Engr. E. F. Hammel, of the Bureau of Buildings, showed a load of 365,000 lb. on each column (R. W. Boyd estimated 396,000 lb.) equivalent to 14,600 (15,850) lb. per sq.in., which is 50% higher than permissible under the Building Code and about the same amount higher than the designed dead- and live-load.

The two tall roof-columns were loaded above balcony level to 166,000 (151,500) lb., equivalent to 7800 (7150) lb. per sq.in. This is higher than intended by the design. The relation of load to probable capacity of these columns is a delicate question on account of their indeterminate bracing. The load at time of collapse was about equal to the designed dead- and live-load.

Part of the over-stressing was due to underestimate of proper load and errors of calculation. Most of it, however, was due to great overloading by *excessive thickness of concrete* fireproofing, slabs and filling.

BRACING OF ROOF COLUMNS (FIG. 4)—Each column was made in two erection pieces, joined by a field splice just below the balcony fascia-girder attachment. The upper section had a shop splice near the upper bracing point. The web of the H-section of the column was north-south, i.e., transverse to the building. In this transverse direction the column was held (between base and cap) only at the balcony level, by the fascia-girder attachment and by an 8-in. I-beam strut running to the wall and intended to be anchored to the wall. R. W. Boyd reports this strut in the case of the north column

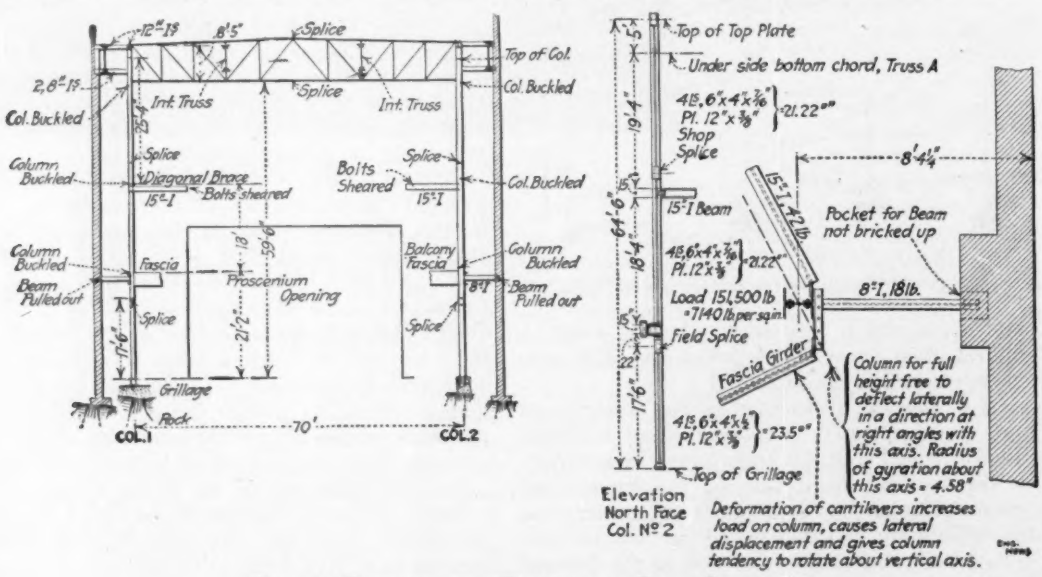

FIG. 4. POSITION AND BRACING OF ROOF COLUMNS.
(Dotted lines in plan of bracing indicate axis said to be least braced.)

FIGS. 5-8. BREAK AND DISTORTION OF TRANSVERSE TRUSS UNDER BALCONY, ORPHEUM THEATER
(South column at left, has been blamed for the collapse.)

as not being bricked into the wall but simply set into a recess.

In the east-west direction, i.e., at right angles to its web, the column was braced at the balcony level (about 19 ft. above its base) by the attachment of the balcony fascia-girder to one flange of the column, through a 15-in. channel connection about 10 in. long. About 19 ft. above this point it was braced forward to the truss over the proscenium opening by a 15-in. I-beam strut; this was about 19 ft. below the roof truss.

CONDITIONS AT BALCONY TRUSS COLUMNS—The balcony truss columns were unbraced between base and cap, and, due to the elastic distortion of the balcony framing, the columns would be subjected to some shift of the top and consequently exposed to bending stresses. The connection of the truss shoe to the top of this column being formed by riveting together the flat shoe-plates and by a splice-plate outside, the column can be regarded as continuous from top of truss to base of column, or 27 ft. long. This column was held to the balcony by its attachment to the balcony truss, by a clip-angle connection to a 15-in. I-beam curb, and by a 12-in. beam to the wall just below the top chord, with connection to longitudinal mezzanine framing.

DEFLECTIONS OF BALCONY—The shallow balcony deflected considerably. The deflection at the ends of the cantilevers amounted to 2 to 3 in. The balcony truss deflected about 1¾ in. at its middle point, and was pulled back similar amounts (the cantilevers pivoting around their rear ends). These various motions exerted pulls on the balcony columns and the roof columns through the connection of these columns to the balcony framing.

CAUSES OF COLLAPSE

The various reports describe the materials and workmanship of the structure as excellent. The column foundations also being found in good shape when the wreck-pile had been explored, there remained for consideration only the three main failures: roof-columns, balcony-columns and balcony-truss. The conditions existing in the structure were such that any one of the three causes might have been the primary happening, and in each case the general result would have been about that displayed by the collapse.

BUCKLING OF THE LONG ROOF COLUMNS

Three of the consulting engineers reporting to Fleischmann Bros. Co. blame the buckling of the roof columns for the start of the collapse. One of the engineers singles out the north column (Col. 2).

Weiskopf & Burroughs state essentially:

> Had these columns been properly supported at about the third-points they would not have failed, and we are of the opinion that good and sufficient supports could have been provided. Considering the fact that the lateral supports for the columns were connected eccentrically, we are of the opinion that these columns had very little, if any, support at these points. . . . . Our conclusion is that incipient failure was due to insufficient lateral support of Cols. 1 and 2 and that the remainder of the collapse followed as a natural sequence. It is, of course, impossible to conclude which column failed first.

L. A. Ball reports as follows:

> In my opinion this collapse was due to the faulty design

of the structural steelwork in not providing sufficient bracing for Cols. 1 and 2. The bracing which was provided was not only inadequate, but increased the tendency of these columns to buckle, and inasmuch as these columns were not adequately braced they were virtually unsupported from top to bottom and just prior to the time of the collapse had reached a point of incipient failure due to the dead-load intensified by the eccentric connection of the balcony fascia support.

The case for the failure of the roof columns is given most fully by R. W. Boyd, who refers specifically to the north column:

At the time of collapse this column above the balcony was carrying a load of approximately 76 tons and was being subjected to a compressive stress of 7140 lb. per sq.in. The connection of the fascia girder with the column added an additional load. . . . . Assuming the 8-in. beam at the fascia-girder connection as of no value as a tie or strut, as was the case (the beam not having been bricked into the pilaster), Col. 2 throughout practically its full height was free to deflect in a direction perpendicular to an axis approximately parallel with the 15-in. I-beam at the level of the proscenium girder. It will be seen that the fascia girder connection and the connection of the 15-in. I-beam provided little, if any, restraining influence against deflection in a direction perpendicular to this axis. The radius of gyration of the column section about this axis, as indicated in the sketch, was approximately 4.58 in. Assuming the column as unsupported from the top of the grillage to the first rivet-hole in the connection for the bottom chord of roof truss, it will be noted that the column had a ratio l/r of approximately 156. Section 138 of the Building Code fixes the safe unit-stress in columns as 15,200 — 58 l/r, limiting the value of l/r to 120. From the figures given above it is apparent that this column as constructed was beyond the limits of the Building Code formula. Applying the formula, however, for purposes of comparison only, we find that the safe stress for the column was 6150 lb. The actual stress (7140) exceeded this figure by approximately 16%. In the case of a column such as the one in question designed with a single web and with a maximum diameter of 12¼ in. and a total unsupported length of nearly 60 ft. and built up in three sections with riveted splices, I am of the opinion that even a ratio of l/r of 120 is excessive, and notwithstanding the provision of the Building Code a column under such circumstances should be designed on a much more conservative basis. With such a column, considerations such as the liability of inaccuracy of fabrication, the danger of damage in handling, and the great difficulty in making two splices so that the axis when erected is a straight line and truly vertical are of much greater importance in the determination of the safe stress than a theoretical column formula based on more or less ideal conditions. Col. 2 under the imposed load was, in my opinion, in a dangerous condition. * * * *

An examination of the construction adjoining the columns and rigidly connected thereto by means of riveted joints reveals a condition which in my judgment provided a considerable disturbing influence and was the immediate cause of failure of one of the columns * * * * Under actual load the ends of the side cantilevers deflected vertically about 2¼ in. and moved horizontally toward the east about ¼ in. The center cantilever deflected vertically about 3⅝ in. and moved horizontally toward the east ⅛ in.† This displacement of the ends of the cantilevers produced considerable movement in the fascia girder to which the cantilevers were rigidly riveted. The fascia girder in turn transmitted this movement to Cols. 1 and 2 to which it was attached by connections having the form and value of lever arms. * * * *

In my opinion the conditions at Cols. 1 and 2: The construction of the columns themselves, their high ratio of slenderness, the excessive unit-stresses, and the disturbing influence of the balcony cantilever, were responsible for the collapse of the theater, the collapse beginning with the failure of Col. 2. * * * *

The fascia girder forming the balcony rail in collapsing was bent back over the overhanging portions of the cantilever trusses. Had the balcony hit the auditorium floor before being struck by the roof it is apparent that this fascia girder would have been bent in the opposite direction. It should be noted in addition that in all of the cantilever girders the beam portion at the outer end shows a tension break in the top chord at its junction with the truss portion of the cantilever, indicating an excessive top-chord tension at some time during collapse. This could easily have been caused by a severe blow at the outer end of the cantilevers when the roof struck the balcony and could scarcely have been caused had the balcony struck the auditorium floor first. * * * *

Chief Inspector Ludwig's report says in part concerning this column:

It will be noted that only one flange of the column was braced, and that very inadequately. If the lower bracing had been in, the column would have been materially strengthened. To all intents and purposes the column had an unsupported length of over 60 ft., and its sectional area should have been determined by that length, instead of considering the bracing provided as effective and using the reduced lengths, as was apparently done.

The least radius of gyration of the column is about 2.54 in., and if the length be taken in round numbers at 60 ft. the value of l/r is found to be equal to 283 instead of 120 as permitted under the Building Code. The actual load on the columns at the time of the collapse as nearly as can be estimated amounted to about 83 tons or 166,000 lb. The sectional area of the column is 21.22 sq.in., hence the load is 7823 lb. per sq.in.

These two columns were apparently the weakest link in the chain, and in consequence many have concluded that to the crippling or buckling of these columns, the initial failure was due. While theoretically it would appear that the columns should have failed under the load, yet nevertheless there is serious doubt as to whether or not this happened. An inspection of the columns after the collapse failed to show conclusively any such crippling or buckling as would naturally be expected. The columns are bent only at points of attachment of the bracing, such as it was; between these points comparatively little distortion was noted. The bends in the two columns are very similar.

Of the several theories advanced, not one is conclusive or satisfies all conditions.

FIGS. 9 AND 10. BUCKLED COLUMNS OF ROOF TRUSS, ORPHEUM THEATER COLLAPSE

(Seen looking east, toward balcony. North column, at left, is shown again on the next page.)

†The horizontal deflection due to pivoting of the cantilever as a whole, as the balcony truss deflected, is evidently not included in these figures. It would increase the horizontal movement to more than 1 in., probably.—Editor.

In spite of this agnostic conclusion, Mr. Ludwig says farther on, in drawing conclusions as to better construction:

If the box type of column had been used in this instance instead of the H type, it is extremely probable that no collapse would have occurred.

With regard to the amount of bracing supplied both here and at the balcony truss columns, he considers that sufficient care had not been given to providing adequate bracing. He does not go into detail on this point, however. Earlier in the report, as quoted above, he says that in his opinion the roof columns should have been calculated as if unsupported laterally between their ends.

To supplement the various opinions on the failure of these roof columns above stated, one consideration must be mentioned that bears on the question whether the roof or the balcony fell first. Attention has already been directed to the fact that the roof columns did not buckle *between* bracing, but at the braced points. This is in itself peculiar, and hard to explain as a primary action. But in any event it means that if buckling of these columns was the first of the sequence, it must have occurred through the lateral yielding at one of the braced points, with the instantaneous development of the buckling at this point into complete failure. Therefore, the corresponding braced point must be found in the wreckage somewhere far out of line of its original location, and torn away from its bracing.

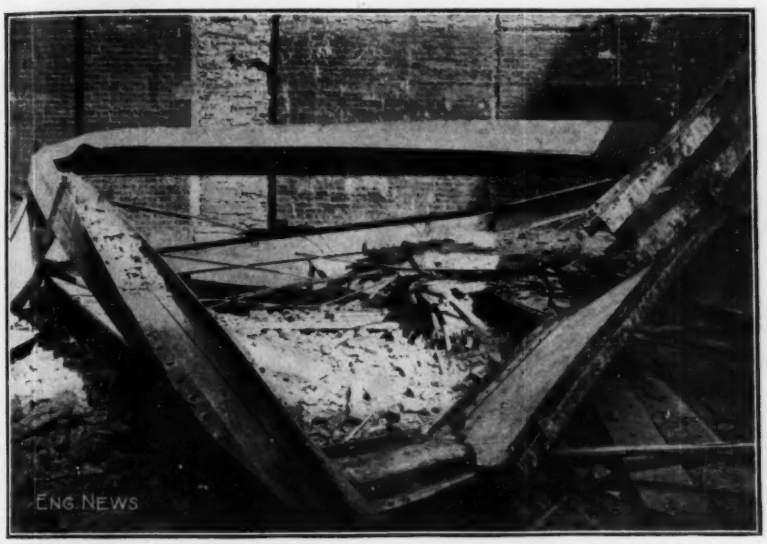

FIG. 11. NORTH ROOF COLUMN (COL. 2)

Refer to the plan, Fig. 3, showing the lay of the principal parts of the wreckage and Fig. 4, showing the points of attachment of bracing to the roof columns. At the upper braced point the column was held by the proscenium strut in one direction and was free in the other direction. The most probable buckling is transverse to the proscenium brace and therefore the point where the proscenium brace is attached to the column should be found far over to the north or south. This is not the case, the point in question lying fairly well under its original location. If the column, however, failed in the east-west direction (i.e., in the line of the proscenium brace), this brace would be pulled far away from the proscenium wall or pushed back into it. Neither event happened, as the sketch fairly well indicates, in either of the two columns. At the lower braced point, where the fascia girder was attached, failure in north-south direction (transverse to the plane of the fascia-girder attachment) would have thrown this point of the column very far north or south. The aspect of the wreckage does not suggest such action. The entire situation, in fact, is out of harmony with the theory of primary roof-column failure, while agreeing with the view that the balcony failed initially and through the attachment of its fascia girder to the roof columns pulled these out of line and brought down the roof.

FAILURE OF BALCONY COLUMN

T. J. Foster (National Bridge Works) finds the explanation of the collapse in buckling of the balcony-truss columns (Cols. 3 and 4). As already mentioned, these columns were loaded to 15,000-16,000 lb. per sq.in. Fig. 12 shows the arrangement of one of these columns with respect to the balcony truss. The H-section of the column and the weakness of the outstanding legs of the angles against local crippling under bending stresses is emphasized by Mr. Foster as well as the fact that the column was not braced except by its attachment to the truss, the truss being braced through its connection to the general balcony floor framing. Deflection of the truss in its vertical plane and movement of the truss backward through the general deflection of the balcony and the pivoting around the rear of the cantilever trusses is held to have subjected these columns to a bending and tipping action initiating failure of the outstanding legs of the column angles just under the bottom chord of the balcony truss. Mr. Foster's statement follows:

The only explanation of the position of the balcony columns with respect to the balcony truss, the top chord of the balcony truss not being buckled, the excessive amount of loose concrete along the south side of the truss, and the break in the bottom chord of the truss showing a break by shock, is that the south column failed first.

In looking for the cause of the failure of the south column, we find that the column was not overloaded in direct compression although the load actually on it at the time of the failure was greater than the estimated load that was to come on the column. We find that the deflection of the balcony truss at its center under its figured load would be about ⅝ in., and under the load actually on it at the time of the failure this deflection was no doubt over an inch. On account of the great slope of the balcony and the rigid connection of the cantilevers at the rear, a vertical deflection would cause almost the same horizontal deflection. The steel was also so thoroughly encased in concrete that this truss probably did not bow as much as if it was not incased, causing the truss to be pulled back for its entire length and causing an eccentric loading on the columns.

There was no bracing of any description to prevent the movement of the steel to the rear. The excess loading on the balcony must have caused deflection. This deflection must have been transmitted to the columns. It could not be transmitted to the columns without the balcony truss moving toward the rear of the building because the cantilevers were rigidly connected to the columns in the rear, and the great slope of the balcony with no bracing allowed the deflection caused by the great weight to push everything back.

That the collapse came when it did may be due in part to the influence of blasting. On the day of the collapse half a dozen or more blasts were set off in the rear part of the

theater, several of them late in the afternoon. The development of strains and deflections in the balcony structure would undoubtedly be hastened by the shocks of blasting. A resulting sudden shift would suffice to bring the balcony columns instantaneously to the crippling point, in view of their condition.

This theory is illustrated by the right-hand sketch in Fig. 12, indicating probable conditions of deflection. The top chord of the truss was so closely associated to the balcony slab that, if the slab just over the column moved back, the top chord must have moved back with it. The bottom chord, however, could twist between column and

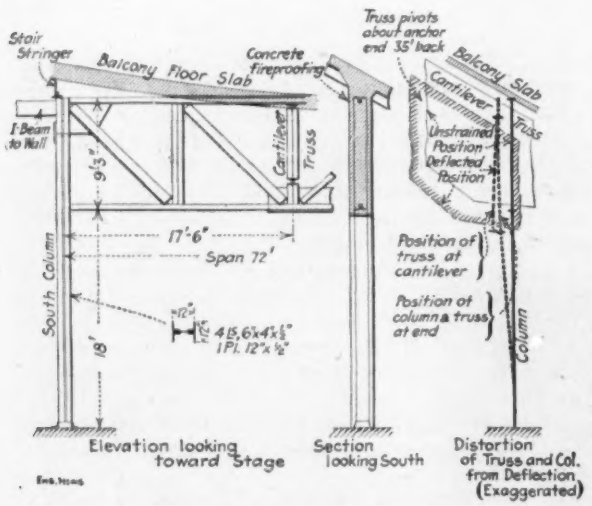

FIG. 12. BALCONY COLUMN

nearest cantilever truss to suit the deflection curve of the column; thus the end portion of the truss became warped, and the column was subjected to a bending moment.

Henry W. Hodge, of the consulting firm of Boller, Hodge & Baird, examined the wreck with Mr. Foster, but did not go into calculations on the design. He also reached the conclusion that the balcony columns buckled first.

R. W. Boyd considers this theory and concludes against it:

> My computation of the actual load on the column at the time of failure indicates that the load was approximately 198 tons, the actual intensity of stress in the column being approximately 15,900 lb. per sq.in. While this stress is greatly in excess of what should be allowed in safe design I am of the opinion that in the absence of a disturbing influence the column could not have failed under this stress. The movements were so minute that they could not have raised the stress in the column to an extent sufficient to have caused failure. * * * * In my judgment primary failure did not occur in either Col. 3 or 4.

Chief Inspector Ludwig disposes of this theory with the following comment:

> The deflection at the center of the balcony truss was slightly less than 1¾ in., and that of the side cantilevers about ¾ in., and this latter, while undoubtedly having a tendency to produce a bending in the column, hardly seems sufficient to have caused the initial failure, especially if we take into consideration the weakness of the attachment of the bottom chord of the truss to the column. It is, of course, possible that the direct load on the column (365,000 lb.) in conjunction with the horizontal component due to bending may have been sufficient, but it seems remote.

In considering the balcony column with regard to length, slenderness and buckling, it is probable that the square seating of the truss shoe on the column, with rivets connecting them, made the entire combination a column from grillage up to top chord. This column, about 27 ft. long, held at the bottom by its anchor bolts and at the top by its close association to the balcony slab, must have failed (if at all) by direct buckling under quiescent load, that was central except for the slight east-west bending produced by the twist of the truss.* Buckling of a short solid-web column at 16,000-lb. loading, with only minor bending, is not on record, so far as we know.

TENSION FAILURE IN BALCONY TRUSS

The report of Thos. J. Bird to the Fleischmann Bros. Co. states that the balcony truss failed in its tension-chord splice, and that this caused the collapse. He gives the following reasons for his conclusion:

> 1. The net area of the splice material in the bottom chord of the transverse balcony truss is considerably greater than the net area of the main section of the bottom chord, proving that the splice material must have been imperfect, or failure would have occurred in the main section before it would have occurred in the splice.
>
> 2. The cantilevers are pulled apart in the bottom chord back of the transverse truss, which indicates that this member was pulled apart by the fall of the transverse truss instead of by any force which might have been produced by the fall of the roof striking the balcony, which case would have buckled this member instead of pulling it apart, or would have pulled apart the top chord instead of the bottom chord.
>
> 3. As the end of the balcony fascia-girder and Cols. 1 and 2 (the roof columns) lay on top of the center part of the balcony fascia-girder, the center part must have fallen first and pulled columns 1 and 2 down afterward.

The last mentioned fact supplements the deductions from the form of buckling assumed by the roof columns, as described earlier.

The theory of the tension-splice failure is discussed also by T. J. Foster and in the reports of R. W. Boyd and Chief Inspector Ludwig. Mr. Boyd gives the stress in the splice as "in excess of 22,000 lb. per sq.in.," and then says:

> Notwithstanding the fact that the truss is badly twisted and that there is a break in the tension splice in the bottom chord, there is every evidence that the failure of this truss was caused by the collapse and not by primary failure in one of its members. A close examination of the material at the broken splice and of the fracture of the top chord angles indicates that the material did not undergo much deformation prior to failure, as would have been the case had there been a critical stress long continued. There is every evidence that these breaks are the direct result of a load suddenly applied, causing very large instantaneous stresses. The falling of the roof onto the balcony would provide all of the elements necessary for the failure of the balcony truss as it actually took place.

In part, the same argument is made by Mr. Foster, but he brings out some further facts against the tension-failure explanation:

> It is practically true that an engineer as well as a layman takes some conspicuous point of failure and builds a defense about it instead of making a general analysis and then finding a cause. One of the conspicuous points is the splice in the lower chord of the balcony truss. The fracture of the steel plates at this point shows no reduction in area or elongation, and shows a crystallized fracture. Throughout the building where breaks occurred by tension there was a good reduction in area, elongation and fibrous metal, while in case of shock the breaks showed no reduction in area and the metal was crystallized. The metal in the broken parts of the bottom chord of the truss was examined and found to be medium steel. If the initial break had been at this point and the structure had given way due to the strain in the metal, there would surely have been a reduction in area, elongation and fibrous metal.
>
> Another reason for this splice not being the cause of the

*The north-south bending due to vertical deflection of the truss need not be considered, because the columns failed by east-west bending.

failure is found in the distribution of loose material resulting from the collapse. As can be seen from a front view, the entire collapse shifted to the south. Wreckage and loose concrete were deposited much more to the south of the center of the building than to the north of the center. We might disregard this as to the roof, but it is particularly true of the balcony truss, along the south half of which the loose concrete was piled much more densely than at any other portion of the building, showing that the loosening concrete was guided in this direction by the sloping steel; the steel could not slope toward this point unless this point went down first. The south wall showed that the truss fell to the south as far as it was able, and some beams which were a part of the gallery and which fell with it prevented the truss going further to the south, these beams making very heavy marks in the brick wall on their way down, cutting a gash in the wall 2 to 4 in. deep. If the initial break of the girder had been in the center the balcony would have doubled in on itself and the two broken ends of the bottom chord would have been found a considerable distance apart and the top chord would immediately have received the balcony load and would have buckled badly. The top chord did not buckle, and the two ends of the broken bottom chord were not over 2 ft. apart, and this is easily accounted for by the drawing back

FIGS. 13 AND 14. THE BALCONY COLUMNS AFTER THE COLLAPSE, SEEN LOOKING TOWARD STAGE
(South column at left; this has been blamed for the collapse.)

of the north cantilever because this cantilever had not yet reached its resting place when the south end of the truss was down and the bottom chord broken.

If the lower chord had parted first it would have tended to pull the tops of the two columns together and to push them farther apart on the line of the lower chord. The north column showed distinctly that the lower chord pulled the column and there is no distortion of this column at its top. The south column failed at the lower part of the truss and perpendicular to the line of the truss, and the flanges of the column at its top directly under the seat of the truss were buckled, also at right angles to the truss. If the truss had parted at the splice at the bottom chord it is inconceivable how the columns could have buckled in the maner described above.

The Bureau of Buildings regarded the question of this tension failure as of sufficient importance to have the material of the splice investigated. Prof. Wm. Campbell, Columbia University, examined samples of the splice material micrographically. The results of his examination show fairly satisfactory material in the splice, except in the lower flange plate. He reports on 8 samples of splice material as follows:

Sample 1. Some lamination and segregation, but not more than is commonly met with in this grade of steel.
Sample 2. Good material.
Sample 3. Segregation and lamination, with the characteristic coarse grain of material finished hot. While not good, the material cannot be classified as bad.
Sample 4. Very low carbon. Parts of it show a good deal of manganese sulphide and slag in thin streaks; same remarks as on preceding.
Sample 5. Similar.
Samples 6 and 7. While of low carbon and of fine grain showed too much segregation and must be classified as bad material, which is confirmed by marked brittleness in the testing machine.
Sample 8. Good material.

The two pieces described as brittle came from the lower splice-plate. The tension specimens in the testing machine showed 20,100-21,100 lb. yield-point and 29,100-27,800 lb. ultimate strength for the first, and 20,500 lb. yield-point and 23,900 ultimate for the second. The section at center of both was close to 0.50 sq.in., but as both broke in the grips the actual breaking stress is not easily stated. They broke without perceptible elongation or ductility, and showed fine crystalline fracture.

Chief Inspector Ludwig concludes against the view that the balcony truss failed first:

If this fracture of the member was the cause of the collapse of the structure, the member must have failed in ten-

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The reference to hard or brittle material needs to be supplemented by a reference to the injurious effect of punching in such material. The splice-plates being punched in close spacing, extensive injury might result in the form of incipient cracking and hardening from the flow of material around the punch. Even steel which gives a good tension test may be thus injured.

Another fact not mentioned in the above discussions is the large amount of the secondary stresses which occur in the chords of shallow trusses. The existence of large secondaries is not merely inferred theoretically, but was proved in strain-gage tests on a New York skyscraper during erection several years ago. A large truss in this building, carrying several columns, was gaged on the top flange of the lower chord to measure the chord tension as erection of the stories above proceeded (the bottom flange was not accessible). The result was that no tension, but at some readings a slight *compression* was found; the bending of the chord, because of its own stiffness, decreased the chord tension to zero in the upper flange and, consequently, doubled the tension in the lower flange. Such action occurring in the balcony truss would increase the computed splice tension of more than 20,000 lb. per sq.in. by a considerable amount.

Following the clearing up of the wreckage of the Orpheum Theater, the structure was rebuilt on the original lines, but with increased sections in trusses and columns, with channels riveted over the column angles, and with added bracing of the columns. The original contractors built the new structure. It was completed in November, 1913, and is now in use.

Bituminous Surface Treatment and Dust Preventives Used in Philadelphia, Penn.*

By William H. Connell†

During the year 1913 the Bureau of Highways of Philadelphia treated 415,000 sq.yd. of waterbound macadam with various tars and asphalt cut-backs, at a cost of 4.9c. to 10c. per sq.yd. The high cost for some of this work was due to several reasons: (1) The work was widely scattered throughout the entire city and in a large number of cases consisted of only one or two blocks, making it very expensive to move the gangs and equipment from place to place. (2) Lack of experienced men to handle the work. (3) The high cost of suitable top dressing available in this locality.

The material used was a torpedo sand passing a ⅜-in. screen, with not over 10% passing a No. 20 sieve, which cost from $2.50 to $2.75 per ton delivered on the road (also the standard price for clean trap rock chips in this locality). The labor cost for sweeping and chipping ran from 0.7c. to 3.6c. per sq.yd., and chips and gravel from 1.8c. to 4.8c. per sq.yd.; bituminous material cost from 1.3c. to 4.5c. per sq.yd. applied. The cost of each of the bituminous materials was as follows, per gal., applied: "Tarvia A" 8.5c., "Tarvia B" 7c.; "Ugite 1-C" 7c., and "Ugite No. 3" 8c.; asphalt cut-back, 12c.

Realizing the importance of cleaning the macadam thoroughly before applying the bituminous material, both hand and machine brooms as well as scrapers were used, care being taken to remove all dust and dirt so that the stone was absolutely bare. After this sweeping the bituminous material was applied in quantities of from ⅛ to ½ gal. per sq.yd.

Wherever possible the road was blocked off and the material allowed to penetrate for 24 hours before chipping. If traffic conditions would not permit blocking the road for this period of time, it was chipped before stopping work for the day. The quantity of chips applied averaged from 13 to 34 lb. per sq.yd., depending upon the amount and character of the bituminous material used.

Every effort was made to avoid creating a pad on the macadam surface, especially on heavy grades. Where the bituminous material was applied during the cold weather it was found that very little, if any, penetration was secured, which resulted in a thin pad. Several old sections previously treated by tar penetration were given a light application of about ¼ gal. per sq.yd. of tar and then chipped, which in most cases has put the road in good condition and will undoubtedly lengthen its life for a couple of years.

The asphalt cut-back consisted of 65% of asphalt having a penetration of 140 to 160 at 77° F.; it was cut back with 35% of commercial naphtha having a specific gravity of 53° to 55° Baumé. This was applied in quantities of from ⅛ to ⅓ gal. per sq.yd. The naphtha acted as a carrying agent, and in most cases a penetration of about ¾ in. was obtained, depending upon the temperature. The material was applied cold, and in some cases was fired immediately after the application, burning out the naphtha, which resulted in an immediate set in the asphalt; and up to the present time the results have been very satisfactory.

The results obtained from the various tars used have been very satisfactory with the exception of a few sections which were applied late in the Fall, and which we believe contained too large a percentage of asphalt in the tar, causing a very mucky condition after each rain.

The primary consideration pertaining to the application, all other conditions being satisfactory, was to avoid a pad, the object being to waterproof the surface of the roadway and use just enough material for this purpose so that the surface would wear off uniformly and be in condition for a new application when necessary, instead of having it creep and pit in spots, which is often the case with a bituminous pad, and which is a condition very difficult to satisfactorily repair. The selection of the bituminous materials used in each case was governed by the fundamental or underlying principles covering this phase of the subject.

The tar treatments were applied hot and cold under pressure, depending upon the material used. In the case of the material applied cold, about ⅛ to ⅓ gal. per sq.yd. was used, while in the case of the hot application from ¼ to ½ gal. was used. The cost of the cold application of tar averaged 6.7c. per sq.yd. while the hot application averaged 8.7c. per sq.yd. These costs are high, notwithstanding the high cost of top dressing in this locality, due to this class of work being new to the organization performing the work.

Other materials used were "Glutrin," which was diluted by adding 50% of water, and applied at the rate of ⅓ gal. of this mixture to the sq.yd.; and calcium chloride and asphaltic oil, of about 18° to 21° Baumé specific gravity applied at the rate of ¼ gal. to the sq.yd. These two latter materials were used simply as dust layers.

Little progress has been made in treatments designed to waterproof and rejuvenate old sheet asphalt and such types of pavements, except possibly in St. Louis, Mo. It is far from exaggeration to say, however, that large sums of money can be saved by prolonging the life of sheet asphalt and asphalt block pavements by suitable surface treatment. The same can also be said of old bituminous pavements built either by the penetration or mixing method, and a great deal of work of this character has been done in Philadelphia this year.

Considerable experimentative and research work, however, has been carried on in connection with bituminous surface treatments on concrete throughout the country without entirely satisfactory results. The city of Philadelphia has constructed a service test road, on the Byberry and Bensalem Turnpike, in which several sections are surfaced in this manner. The first report on the test road, dated November, 1913, is now available.

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Editorials

The cutting off of the water-supply of the greater part of the city of Montreal on Dec. 25 (see a later page of this issue) illustrates anew the danger hanging over every city which depends upon a single water-supply conduit from an outside source of supply and has no or practically no reservoir storage at the city end of the conduit. Montreal has had numerous warnings of this danger during the past half-century, both through accidents and engineering reports.

The various organizations which agitate in favor of large national expenditures on waterway improvements have usually set their goal at an annual expenditure of $50,000,000 upon such works. It may not be generally realized that we are already within measurable distance of such an annual expenditure. The annual report of the Chief of Engineers just issued shows the total expenditure by the Federal government on all classes of river and harbor work during the fiscal year ending June 30 last, to have been nearly $42,000,000. For the fiscal year ending June 30, 1915, the Chief of Engineers submits estimates amounting to nearly the same sum. It is of interest, also, to compare the present annual rate of expenditure with the expenditure in previous years. The total of all appropriations by Congress for river and harbor work of every sort since the foundation of the government up to June 30 last, was under $792,000,000. Thus the present annual expenditure on river and harbor work is about one-nineteenth as much as all that has been expended by the government on this class of work up to the present time.

The reappointment of Rudolph P. Miller, M. Am. Soc. C. E., as Superintendent of Buildings of the Borough of Manhattan, New York City, has just been announced by Marcus Marks, the new President of that borough. Before Mr. Miller's appointment by Borough President McAneny four years ago, the office that he holds was a political plum to be awarded to the faithful and its conduct bore witness of the fact. Mr. Miller's broad experience as a structural engineer and his inside knowledge of the department gained while its Chief Engineer served jointly to govern his policies and methods and happily resulted in an admirable administration. The city has been served as never before and the builders, architects and engineers doing business before the department are filled with a rare satisfaction. The reappointment should be pleasing to all friends of good government and particularly to engineers. Mr. Miller's service is an example of the conspicuous success of an engineer in an office essentially engineering in character but customarily filled by the layman, politician or business man.

The results of the activity of engineering societies, engineering periodicals and individual engineers in speaking up for the engineering profession are beginning to be made evident. During the past year we have noted in our personal columns the appointment of several engineers to public-service commissions, the election of engineers as commissioners under the new commission plan of city government, the selection of an engineer to be city manager of a large Ohio city, and now to cap the climax for the year 1913, we record with pleasure and satisfaction the election of a city council of three commissioners, each and every one of whom is an engineer by training, experience and practice. Lethbridge in Alberta, the first city in Northwestern Canada to have a commission-plan charter, is the fortunate municipality. Its charter provides for a Mayor, who is also the Commissioner of Finance and Safety, at a salary of $4000 per annum, a Commissioner of Public Works and a Commissioner of Public Utilities, each at $3500 per annum. The first, as noted elsewhere in our personal columns, is a civil and mining engineer, the second a civil engineer and the third an electrical engineer.

A larger degree of reason is shown in the decree in the Grand Rapids sewage-disposal case, handed down late in December, 1913, than appeared in the original decision of the State Supreme Court. In the decision the city was apparently required to install a septic tank; in the decree the city was to stop discharging its sewage into the Grand River "until the same shall have first been so deodorized and purified as not to contain the foul, offensive or noxious matter (which it now contains) capable of injuring complainants or their property or causing a nuisance thereto." In leaving to the city of Grand Rapids the selection of the means to be provided to stop the nuisance which seems to have been proven by "the people of Wyoming township" the court has wisely refrained from setting itself as an expert in sewage treatment and at the same time falling into the common popular notion that the septic tank is the sum and substance of sewage treatment and the cure-all for stream pollution.

The Admission of Civilian Engineers to The United States Engineer Corps

Great hopes were entertained two years ago that the United States Engineer Corps might hereafter be recruited by picked students from other engineering schools as well as from the graduates of West Point. The need of numerical additions to the Corps of Engineers had been repeatedly brought to the attention of Congress. By a law passed in 1911, twelve vacancies were created in the Corps which could be filled by appointment from civil life, and twelve additional vacancies have occurred, in accordance with the terms of the law, in each of the two succeeding years. At the present time, according to the report of the Chief of Engineers just issued, there are

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Great hopes were entertained two years ago that the United States Engineer Corps might hereafter be recruited by picked students from other engineering schools as well as from the graduates of West Point. The need of numerical additions to the Corps of Engineers had been repeatedly brought to the attention of Congress. By a law passed in 1911, twelve vacancies were created in the Corps which could be filled by appointment from civil life, and twelve additional vacancies have occurred, in accordance with the terms of the law, in each of the two succeeding years. At the present time, according to the report of the Chief of Engineers just issued, there are

23 vacancies in the grade of Second Lieutenant, which according to the law may be filled by appointment from civil life. This law, however, remains practically a dead letter. During the year ending June 30, 1913, to quote from the Chief of Engineer's report, "only one probational second lieutenant was appointed to the Corps from civil life;" and so far as we are aware, nothing whatever is being done toward filling these 23 vacancies with civilians.

As many of our readers will recall, this law when finally enacted was a serious disappointment to the engineering profession. When originally introduced it was hoped that it might open the way by which the men of long experience and great ability among the U. S. Assistant Engineers might receive recognition and reward by appointment as commissioned officers, while at the same time the government would attach them to its service so that their ability and experience would be permanently at the government's command. Before its final passage, however, the bill was emasculated so that in its final form it provided only a means by which recent engineering graduates might receive appointment as second-lieutenants.

The above quotations from the report of the Chief of Engineers make it evident that the law has been further emasculated in its execution so that its purpose has not been carried out. It will be interesting to the profession to know the reasons for this. The need for a larger number of officers in the Engineer Corps has been many times emphasized, and becomes more and more pressing with the constant increase in the duties which are laid upon the engineers who are members of this body. Congress has made provision in the law above referred to for the desired increase; and while the increase is limited to the lowest grade, there can be no doubt that the young men appointed to this grade will be greatly needed in their country's service five and ten and twenty years hence. It is unreasonable to suppose that among the thousands of graduates yearly turned out from our best engineering colleges there are none sufficiently qualified to be eligible for appointment to these vacancies. The question recurs, therefore, why for two whole years nothing whatever has been done toward filling these vacancies in the corps.

The Structural Engineer and the Orpheum Theater Failure

Steelwork design is so precise and definite, especially in the case of buildings, that the story of a failure is rather disconcerting reading. The engineer justly feels certain of his data, in building matters; so that, possessing clear and direct methods of computation, and knowing that the object of design is free from disturbing incidentals such as impact, he relies on the result with perfect assurance. What then if a frame collapses when still new, or even before being put into service, like that of the Orpheum Theater in New York? It is necessary to look into the circumstances very closely, and single out the elements of trouble, for future guidance.

In the case of the Orpheum this has been done quite elaborately, with results as set forth at considerable length in this issue of ENGINEERING NEWS. Garbled and halting accounts of the matter having been published elsewhere, there is the more reason for a detailed presentment. Careful study of the facts and deductions which we reproduce can supply both beginner and mature engineer with the groundwork of a good education in structural design. To mention only one or two points of this side of the case: The Orpheum collapse exhibits, more graphically than any previous case known to us, the close inter-relation of strength and elastic nature in most structures, even in such as are strictly "simple (in the limited present-day meaning of this term). In other words, it shows how necessary to the structural engineer is the constant appreciation of strain effects in structures (in contrast to the usually taught view that in practical structures the distorted and initial configurations can be taken as identical, and that strain effects therefore are negligible). Also, it illustrates a useful application of deflection calculations. And, for the more conscientious, it suggests the desirability of brushing up on secondary stresses, which may demand more consideration in the future than they have received hitherto.

Unfortunately, the investigations of the Orpheum collapse did not lead to a unanimous verdict. Three opposing conclusions are presented. One of these centers on simple buckling of a long, insecure column; while this is more or less commonplace, the circumstances are made unusual by the probability that the bracing of this column was the very thing that pulled it down, and that the column would have been safer if it had been kept free from bracing and made to depend on itself.

But the other two versions of the collapse are distinctly new, and highly important:

If the short and stiff balcony columns failed initially, with but 16,000 lb. unit-load, the record of column failures is enriched by a unique figure. Earnest consideration will have to be given to going back to ultra-conservative standards of column design.

If the bottom-chord splice of the transverse balcony truss tore apart, without shock, without external disturbance, just by itself, the tenets of structural engineering are jarred by the first distinct tension failure at a safe stress. And if, as is hinted by the facts, this tension failure is chargeable to the combination of (1) brittle splicing material, perhaps hardened and haircracked by punching and riveting strains, and (2) unsuspected secondary girder-bending of a long, shallow truss, then certain reforms of practice both in design and in shopwork are indicated. In designing, shallow trusses will need more liberal proportioning, in default of analysis of secondary stress. In manufacture, the quality of "minor" parts will have to be looked after, since these very parts are most in need of toughness and deformability.

The introductory "if" of the last two paragraphs may as well be omitted, however. Which one of the three explanations of the collapse is the true one is immaterial, since the six expert statements taken together mean, in effect, that any one of the three primary failures could have happened. Apparently, three separate points of fatal weakness existed. This means that the reform measures which the three explanations severally suggest will *all* have to be considered.

Besides inducing various sober reflections on the part of steel designers, the Orpheum collapse preaches a neat little sermon on the mischief that the over-zealous concrete man can do.

The Increasing Use of the Day-Labor System in Constructing Public Works

Among the many radical changes in public opinion and public customs which the past quarter century has witnessed, one of the most important to engineers is the change which has taken place with respect to the conduct of public engineering works. A quarter of a century ago it was the rare exception to have public work done in any other way than by contract. At the present time a very considerable proportion of engineering construction on public works is carried on by the direct employment of labor without the letting of other contracts than those for necessary supplies.

The change has been most marked, probably, in the works carried on by the Federal government. A quarter of a century ago practically all of the dredging work done by the government in connection with river and harbor improvement was let to contractors. The difficulty in letting contracts for maintenance work, however, where the amount of work and the time involved were difficult to determine in advance and where there were difficulties also in measurement of the amount of work accomplished, led to the purchase and operation of dredges by the government engineers themselves. It was an easy step from this to the use of these same dredges on new construction work. At the present time, while the government still lets to contractors a large amount of dredging, the government itself owns and operates by its own forces what is probably by far the largest fleet of dredges under any single ownership anywhere in the world.

What is most significant, however, is that this radical change has gone on practically without opposition. Engineers have found themselves free to undertake by their own forces, without the letting of contracts, a great amount of work which twenty-five years ago they would have been wholly unable to execute in that way. At that time the river and harbor appropriations were regarded not merely as distributions of "pork," to be equitably divided among the Congressmen from various districts in proportion to their log-rolling abilities, but as the legitimate spoil of the contractors in each district, who were not seldom prominent figures in local politics, with an influential voice in sending a man to Congress and keeping him there.

Under these conditions it can readily be understood that an Engineer-officer who attempted to do by day's work an important piece of public work, having large possibilities of profit, instead of letting it to contractors, had to fight against all the political influence that the Congressman from the district in which the work was located could exert. So while competent engineers were then, as at the present time, well aware of the great advantages of doing certain classes of work by day labor instead of by letting contracts, they were in most cases practically powerless to carry out work in that way.

It must be admitted, too, that the conduct of work by the direct employment of labor was much more difficult for an engineer at that time than it is today. There were then comparatively few engineers to be had with sufficient experience in handling men and materials and organizing and directing forces to enable work by day labor to be conducted with any certainty of economy.

A stock argument of those who opposed the day's-work plan of operations was that the engineer was never a "practical" man. He was recognized as a sort of necessary adjunct to make surveys and to design and plan the theoretical part of the work; but the contractor with practical experience was claimed to be absolutely essential to transform the engineer's theoretical ideas into actual economical construction.

It may freely be admitted that there was in many cases a good deal of ground for this argument. A great change has taken place in the situation, however, during the past quarter century. For one thing the contractor's work has been placed on a far higher plane. It is fully recognized nowadays that for the economical conduct of a contractor's work the advice and supervision of an engineer are essential. At first the engineer was brought in as a mere assistant and adviser of the contractor, to help him to cope with the engineer for the owner even-handed; but the contractor's engineer, if of the proper caliber, soon demonstrated his capacity to go far beyond this; and very often he became the superintendent or general manager for his employer, or he branched out into business for himself. At the present time a large proportion of the successful and reputable contracting firms are made up of engineers of ability and high standing.

All along down the line in a contractor's organization are to be found men with more or less education and experience in engineering work; and as they are working at wages for a contractor, they are equally available for employment on a work which is to be executed by day labor. The engineer at the head of a piece of work, therefore, who decides to perform it by days' work has a much easier task in creating an organization for carrying it on than he had a quarter century ago.

Meanwhile public opposition to the day's-work plan has practically disappeared; or, to express it more accurately, the influences which twenty-five years ago were all powerful in compelling the well nigh universal use of contracts are now opposed by a very widespread public opinion which strongly favors the day's-work plan of operation.

It is true that in many cases the engineer in charge of work has to yield to one or the other of these opposing forces and has not an entirely free hand as to whether work shall be done by contract or day labor. He is often, however, enabled by reason of this conflict in opinion to gain a hearing for his own views and advise as to which plan should be adopted in a particular case.

We express not merely our own views but the views of the engineering profession at large, we believe, when we say that those who favor contract work and those who favor the day's-work plan are each of them partly right and each of them partly wrong. The rule has many times been laid down in these columns that where a public work involves great elements of uncertainty which cannot be foretold in advance, so that the contractor who bids on it must undertake a large gambling risk, and where the work is also of a comparatively simple character so that it can be carried out by such an engineering organization and plant as a competent engineer can bring together, it is under these conditions very greatly for the public interest that the work should be done by day labor and not by contract.

On the other hand, the evolution of the contracting business during the last twenty-five years has made other

changes in conditions. The principal qualifications of a large proportion of the contractors who flourished a generation ago was the ability to handle workmen (the handling being not seldom by methods which would not pass muster at all at the present day) and the possession of some political pull. At the present day the typical successful contracting firm has first of all a thorough organization, built up as the result of years of experience and able to bring efficient management to any task that it undertakes. In the second place it has usually a large investment in uptodate plant and machinery, and a force of men thoroughly experienced in its maintenance and operation. A large proportion of such contracting firms have specialized in some particular class of work and they have accumulated a knowledge and experience that makes them an invaluable ally of the engineer in charge of such work.

It cannot be denied that work in the hands of such a contractor can be executed with an economy and efficiency which is very difficult to equal in an organization brought together by the engineer in charge for use on this particular work alone. The introduction of machinery especially has tended to make contract work more advantageous. The Federal government may, as we have noted above, operate its own large dredging fleet because it is carrying on such work permanently year after year; but where a State or a City undertakes to do its own dredging on construction work, it would have to make a large investment in a dredging outfit which would remain on its hands when the work was completed.

On maintenance work, however, there has been an enormous increase in the use of the day-labor system by Cities and States compared with the practice of a quarter century ago. An example is the maintenance of asphalt pavement, which is now done by most cities with their own equipment where twenty-five years ago [illegible] work was let by contract. In such work as the collection and disposal of refuse also the tendency is very strong to break away from the contract system and have the work carried on by the City itself.

We believe the engineering profession may well congratulate itself that the change in public opinion in favor of the day's-work system of conducting public works is in itself an evidence of the greater esteem in which the public holds the engineer. Every successful engineering work which is carried out by engineers in this manner, without the intervention of contractors, tends to increase this public confidence and raise the engineering profession in public esteem.

It would be difficult to over-estimate the value in this matter of the work on the Panama Canal. Most of our readers can recall the brisk controversy that raged half a dozen years ago over the question of letting the work to contract. The newspapers were filled with declarations that only by putting the work in the hands of experienced contractors could it be accomplished in a reasonable time and at a reasonable cost. It was the opinion of many that the strong pressure to turn the work over to contractors could not possibly be overcome. It is probable, indeed, that the work would have been let to contractors except for the great difficulty in drafting any form of contract that would not on its face present an enormous fortune to the contracting firm.

The magnificent efficiency with which the work has been conducted by Col. Goethals and his able staff has been the greatest object lesson the United States has ever had as to the possibility of carrying on public work in the best and most economical manner without any resort to contractors. It would be difficult to overestimate the value of this lesson both to the public and to the engineering profession.

Letters to the Editor

The Sphere not a Prismoid

Sir—In reply to the query, "Is the sphere a prismoid?", you say that "the sphere is enough of a prismoid to answer the formula's requirement."

A generally accepted definition of a prismoid is a "volume generated by a right line generatrix, moving on the bounding lines of two closed figures, of any shapes, which lie in parallel planes, as directrices."

What the "Century Dictionary" calls August's formula, is directly applicable only to such sections of a cone and cylinder, for instance, as conform to this definition. In ENG. NEWS, Apr. 6, 1905, p. 362, appeared an article "proving the applicability of the formula to a large variety of solids, the areas of the transverse sections of which are expressed by an equation of the third degree."

According to Trautwine "the extended application of the prismoidal formula (to the sphere and spherical segments, among others) was first discovered and made known, by Ellwood Morris, a civil engineer, of Philadelphia, in 1840."

The general demonstration given in the article on Mensuration, in the "Encyclopedia Britannica," shows this type of formula applicable to the sphere, ellipsoid, etc., not because their "forms approach the form of a prism," for obviously such solids of revolution are not included in the above definition of a prismoid, but because "the area of every cross-section parallel to these planes is a quadratic function of the distance of the section from a fixed plane parallel to them," including the prismoid as a special case.

For, "the sphere, for instance, whose radius is R, the area of the section at distance x from the center is $\pi (R^2 - x^2)$, which is a quadratic function of x." The areas are, therefore, O, πR^2, and O and the volume

$$\tfrac{1}{6} \times 2R \times 4\pi R^2 = \tfrac{4}{3}\pi R^3.$$

R. H. M.

Medina, N. Y., Dec. 13, 1913.

Concrete Mixing on the Lake Spaulding Dam

Sir—I note the editorial in your issue of Dec. 4, 1913, on "Should Concrete Be Mixed Longer Than Is Now Common," and agree with you that not enough attention is given this part of the work.

On the Lake Spaulding Dam, which is evidently the specific job referred to in the editorial, the 2150 cu.yd. was not run in 8 hr. as you quote, but in 10 hr. Furthermore, the concrete was run in chutes several hundred feet and dropped nearly 150 ft. in baffles in towers, which very materially improves the mix. We made tests of time of mixing and proved that about 1½ to 2 min. is required where there is no subsequent drop through baffles, and a batch in 2 min. was the usual run where the concrete did not get subsequent mixing by dropping through baffles. Two minutes was used as the standard mixing time to give good results on the work.

The time required to mix concrete depends some on the material, whether gravel or crushed rock, and also on the amount of cement; the more cement the more time is needed to mix the concrete and the more time we can afford to give to mixing.

F. G. Baum.

Chief Engineer, Hydro-Electric Dept., Pacific Gas & Electric Co.

San Francisco, Calif., Dec. 7, 1913.

The Need of an Engineering Examination to Determine Clear Titles to Land

Sir—A prudent man, when buying real estate, requires assurance that the title he is to acquire is clear. To this end he employs a lawyer who carefully searches the records to ascertain whether each successive purchaser of the property has duly sold the same, and that no mortgages or attachments are on record against it. Finding such requirements satisfactorily answered, the lawyer usually vouches for the title; but one of the most dangerous defects in the title can only be detected by an engineer or a surveyor, and that is an error in the description, copied through successive deeds, or an encroachment by an adjoining owner whose deed may read as clear as that of the attorney's client. A few instances have recently come to my knowledge which forcibly illustrate the necessity of joint examinations by lawyer and engineer to fully assure the purchaser of safety in his investment.

A city banker bought about 20 acres for a country home, after a thorough title examination by a leading attorney. His land bounded on the principal street and on the town line, and behind him, also bounding on the town line, another party owned to a railroad. The combined distances called for in the two deeds, along the town line, from street to railroad, exceeded the actual distance by about 150 ft. or, in other words, the two parcels overlapped by that much, and about 10% of the banker's purchase had a serious cloud on its title. There was no way for the lawyer to discover this situation from the records, and the writer became aware of the discrepancy while supervising the preparation of plans for the town assessors.

Another instance in the city of Boston: a narrow tract of land was laid out in house lots in 1882; one street extended the length of the parcel and the surveyor made an error of several feet in the length of it, so that the plan did not match the ground. Later, certain purchasers of lots at one end carried a dispute as to bounds into court. A decree resulted by which the length of the first two lots was increased by more than eight feet; but no decision was made as to who should suffer the corresponding loss. No comment is made on the wisdom of the court's decision, but the situation now is that a number of lots have been, and are being, sold by reference to a plan which was wrong in the first place, and which has since been so mangled by the court that no one can tell what it means. The writer, being asked by a personal friend residing on the street, for advice regarding his bounds, discovered the above conditions by an ordinary investigation; but the friend's lawyer, finding a succession of deeds reading alike, and the plan referred to on the records, with no mortgages, liens, or attachments, and taxes paid, reported the title clear.

In some parts of rural New England, the writer has observed a custom of having lands surveyed when passing title. This being usually done in the presence of adjoining owners, amounts to a pretty thorough engineer's examination, and shows the plea, advanced herein, is not a radical innovation.

Fred. J. Wood.

57 Westbourne Terrace, Brookline, Mass., Dec. 15, 1913.

Comparative Cost of Public and Private Irrigation Projects

Sir—In Engineering News of Aug. 21, appeared a long article and several tables under the signature of John E. Field, State Engineer of Colorado, on the Cost of Reclamation Service and Other Irrigation Projects. From voluminous allegations and tabulations, Mr. Field finally evolves his essential conclusion that "The original cost of private enterprises is about one-half that of Government enterprises."

By this, as shown by the context, Mr. Field means that the Reclamation Service as conducted requires twice as much money to do a given amount of work, as exactly the same work would cost if carried out by private enterprise. He claims to know this from personal experience and makes numerous statements of alleged facts to explain his conclusion. This is a very severe arraignment and, if true, would be sufficient to condemn the skill and ability of those responsible for the Government work.

Proceeding from a State official with experience both in private and public work, these statements are being given great weight by the Eastern press, and many clippings come to my desk quoting his conclusions and using them to condemn not only Government irrigation work but all public construction and management of public works. It seems necessary, therefore, to request space for a reply, pointing out some of Mr. Field's fundamental errors.

A typical illustration in the statement in this article that "the water-supply is questionable on the Milk River, Sun River and Shoshone projects," of the Reclamation Service, and he concludes from this that the

acreage on these projects will have to be reduced and the price increased.

The Sun River and Milk River projects were of indefinite extent at the time of the Army Board report and the acreage of ultimate development remained to be fixed according to the water-supply, which could be done without seriously affecting the cost and will be done when necessary in the light of water-supply records available at that time. A recognition of this fact by the Army Board, Mr. Field makes the basis for a condemnation of the water-supply.

In Table 3, he prints as a quotation from the Army Board report concerning the Shoshone project—"area may be largely reduced account water-supply." The Army Board made no such comment. Its "summary" on p. 154 of the printed report says:

> 3. The water supply is sufficient for the whole project.

On p. 197, paragraph 9, the Board says of this project:

> Even asuming all prior rights to be perfected it is found that there will be sufficient water supply for this project.

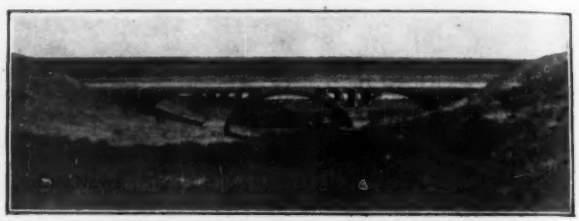
In Table 1, Mr. Field has listed a number of private irrigation projects and given what purport to be the costs. I have not had opportunity to check this table in detail, but some points show with glaring distinctness. For example, he condemns as too high the price of $45 tentatively given by the Reclamation Service for the Greeley Poudre district. This includes what is known as the Laramie Poudre district and I am reliably informed that the present issue of bonds is $5,106,000; that the bonds not sold are held as a bonus by the promoters, and that no water has as yet been delivered to the lands. One of the past officers of the company states that it would take at least $3,000,000 to complete the project. The company has been unable to sell the additional bonds and the project seems to be abandoned. The final cost will probably exceed $60 per acre, if it is ever completed.

He also criticizes the $45 tentatively given for the Denver Reservoir and Irrigation Co. I have not had opportunity to investigate this closely but it is understood that $8,000,000 of bonds have been issued and the project is not completed. The latest reports state that the company is in the hands of a receiver and there are many unpaid bills. On the basis of the present bond issue, as reported to me, and the acreage given by Mr. Field, the cost per acre would be more than three times that given in his table, and the project is not nearly complete. It is also reported that the contracts made by this company call for but five-eighths of an acre-foot of water per acre, which is less than half of a supply.

He has omitted entirely the cost of maintaining and operating many of the canals listed, including the canal of the Orchard Mesa Irrigation District. As this canal was built by the firm of which Mr. Field was a member, he is, of course, familiar with it, yet he understates the construction cost, and omits the cost of operation and maintenance.

This canal contains over four miles of wooden flume on a sidehill, which has caused great expense for maintenance. The County Treasurer's office at Grand Junction shows bonds issued to the amount of $1,075,000, and additional warrants to make up an excess construction cost of $254,480. The County Assessor's office gives the area assessable at 8670 acres, showing a cost for construction of $153.34 per acre, instead of $90 or $[illegible] as shown by Mr. Field's table. The operation and other current expenses for the year 1914 have been estimated by the Board of Directors as $22,000, or over $2.[illegible] per acre. In addition to this, it is reported that there must be paid next year $64,500 interest on the bonded indebtedness and there is also a deficit for past years of $10,000, which will have to be made up. These three items will make a total levy for 1914 of nearly $12 per acre, without paying anything on the bonded indebtedness.

Mr. Field omits entirely all figures of cost for the Colorado Coöperative Company and explains this in the note by a statement that the building of the ditch was a "secondary" matter and that the present secretary of the company cannot give an estimate of the cost, but gives the present value at $33 per share covering two acres. I have before me the financial statement made by the company for last year, which shows that the ditch is carried among the assets at a valuation of $319,466.54, constituting about 95% of the assets and amounting to more than the total capital stock of the company. This hardly looks like a "secondary" operation and the figures



Spring Cañon Flume, North Platte Project, U. S. Reclamation Service

carried in the assets are doubtless the cost as shown by the books. If so, it makes an acreage cost of over $69.

The writer has not had opportunity nor would it be profitable to go further into the looseness of Mr. Field's figures and the inaccuracy of his statements, but enough has been cited to show that by such a method it is possible to reach any desired conclusions from an array of complicated facts.

Mr. Field not only wrongfully states that certain Reclamation projects have a short water-supply, but he entirely ignores this question with reference to private projects. One irrigation engineer, well informed on the subject and now a business partner of Mr. Field, once stated that there was hardly an irrigation project in Colorado that had not sold water rights far beyond its ability to deliver water. It is common knowledge that adjudicated rights to the waters of the Arkansas and South Platte Rivers held by existing ditches amount to many times the available water-supply; in the case of the Arkansas, probably more than ten times. It is likely, therefore, that many of these private projects, which he has listed as being cheap, have a short water-supply. On this point Mr. Field is silent.

The Reclamation Service builds its lateral system to the vicinity of each farm unit, leaving but little, if any, construction for the irrigator, except upon his own land. In addition to this, on many of the projects the price quoted by Mr. Field includes extensive drainage works which are seldom, if ever, constructed by private companies. Mr. Field makes no allowance for these expenses, although to be fair he would have made such al-

lowances at three separate places in his article; first, where comparison is made of cost; second, where he compares the advantages to the settlers on different classes of projects, and third, where he undertakes to discuss "Causes of High Cost of Government Projects." Mr. Field says on p. 352:

> It is the writer's opinion that the Government has thoroughly spoiled the water users by its paternalism and assumption of the burdens properly belonging to the users, who could maintain and operate their lateral ditches, at least, both cheaper and more satisfactorily to themselves than can the Government.

Mr. Field is very solicitous lest some of the public projects may not return their entire cost and, regardless of the facts, takes the responsibility of increasing the estimated cost on a number of public projects for this reason. It is notorious that the majority of recent private and corporate irrigation projects have been financial disasters to the investors; but he makes no increase in his figures for *this* reason.

It can hardly be doubted that had Mr. Field applied the same rigid rules of water-supply and return of cost to private projects that he has applied to public, and had made due allowance for the better character of work, the more complete construction of lateral systems and drainage works and had confined his statements to the facts concerning projects built within the last ten years, as the Reclamation projects have been built, he would have found a higher average cost per acre for private than for public projects.

It is not claimed, however, that this proves or even indicates that work cannot be done as cheaply or as well by private enterprise as it can by public. Each method has some minor advantages and some minor disadvantages and fair comparison can be made only of cases which are exactly similar in physical difficulty; toward this Mr. Field has made no attempt.

During the early history of irrigation, farmers and groups of farmers, naturally confined their efforts mainly to diverting small streams in smooth valleys where the slope of the country and the topography were such as to make the work easy and cheap. With the values of land then existing, no expensive development was practicable. Such development, proceeding for nearly half a century, irrigated in the aggregate a very large area of land. The farmers employed the cheapest class of construction and seldom counted their own time in computing costs, which are hence reported very low, and these appear in the census averages quoted by Mr. Field, in comparing unfairly the cost of private work with the difficult work later carried out by the Government.

As land values increased and the easier projects were developed, more and more difficult ones were taken up, sometimes successfully and sometimes not. As the more difficult problems were attacked, corporate capital and the district system were employed and such projects as they could handle were gradually developed. The inherent difficulties, however, did not admit of much profit to the investor. In fact, in the majority of cases the investors lost a large part of their capital, to say nothing of interest and profits.

This experience made it more and more difficult to enlist private capital in further irrigation enterprises and the increasing difficulty of carrying out many large projects led to the passage of the Reclamation act for the avowed and widely heralded object of enlisting national funds for the development of projects not feasible for private, corporate, district, or State enterprise. This policy, avowed in Congress and announced over and over again by President Roosevelt in public talks, was followed by the Reclamation Service.

Many of the projects undertaken by the Reclamation Service had been outlined by private or local public interests, who had endeavored for years to enlist private capital in their construction. To this class belong the following projects, since taken up by the Reclamation Service:

Boise project in Idaho.
Minidoka project in Idaho.
Sun River project in Montana.
North Platte project in Nebraska and Wyoming.
Truckee-Carson project in Nevada.
Rio Grande project in New Mexico and Texas.
Yakima project in Washington.

The Yuma project in Arizona and California was the successor of several unavailing attempts to place portions of the same land under successful irrigation. The Gunnison tunnel scheme, Colorado, had been agitated for many years and efforts failed to raise private capital. Attempts were made to have it constructed with State funds, but without success.

Shortly after the passage of the Reclamation act, preliminary surveys were taken up on the Grand River project in Colorado. The data collected were considered by the Board of Engineers, who recommended the project as promising and requested further surveys and data. Representations were made, however, that this project could be built by private capital or under the district plan and for this reason the Reclamation Service suspended operation and withdrew from the Valley for the announced purpose of permitting its construction by private capital. At the request of interested parties the Reclamation withdrawals of public land were continued for the purpose of holding these available for reclamation, with the understanding that they would be released in case private capital were enlisted in construction. After five years of effort the citizens of the Valley petitioned Secretary Garfield to take up the project as a Reclamation Service project, representing that private capital could not be induced to enter the field.

In the meantime, an irrigation district had accomplished some partial development by which a small portion of the best land situated nearest the water-supply had been brought under irrigation and the project rendered far more difficult by the elimination of some of the most valuable lands and the increased value of the right-of-way necessary for the canal. Even with these increased difficulties, the project was taken up by the Reclamation Service about a year ago and is now under construction.

Shortly after the passage of the Reclamation act, a very attractive project for irrigating the Billings Bench, Montana, was presented to Director Walcott and Chief Engineer Newell, with an urgent request that they construct this and they were shown over the country for that purpose. The reply was that this was obviously a project so attractive that there would be no difficulty in constructing it under the provisions of the Carey act with private capital and that it did not come within the purposes of the Reclamation act and could not be considered. The project was actually constructed by private capi-

tal and is one of the most successful Carey act projects in the State.

The Fort Custer project on Big Horn River has a similar history, as have many other projects.

The Government early made surveys on the Sun River project in Montana and this was one to which attention was turned soon after the passage of the Reclamation act. It was represented, however, by State officials that this project could be built by private capital under the Carey act and they preferred the public funds to be devoted to other regions such as the Milk River-St. Mary project, less attractive to private enterprise. The Reclamation Service accordingly withdrew from the field. After protracted efforts to finance this project under the Carey act, the Reclamation Service was informed that it was impossible to secure capital for this construction and was requested to construct it as a Reclamation project. Although in the meantime the cost of construction had greatly increased, the project was taken up and is under construction, about 16,000 acres being now under ditch.

Nearly all of the projects undertaken by the Reclamation Service had been abandoned after unsuccessful attempts to finance them as private projects, or else were new projects so difficult as not to attract even the attempts of the promoter. All, therefore, were projects unattractive to private capital.

Manifestly, all those projects actually constructed by private enterprise are from the nature of the case of a character that could attract and secure private capital. If Mr. Field had been able to prove fairly that Reclamation projects have cost more than private projects constructed at the same time, he would simply have proved the truism that *projects attractive to private capital are cheaper than those which are not.*

In practically every case above listed, where extended efforts have been made to enlist private capital, the project finally outlined by the Reclamation Service is more comprehensive than that proposed by private capital; and here rests one of the chief causes of the alleged high cost of Government projects, and the chief justification for the use of public money for this purpose. In many cases, a choice portion of a large feasible development is carried out by private capital because it affords greater promise of profits than if consideration were given to the largest feasible development of the available water-supply.

These controlling facts, well known but often forgotten by the general public, but still better known to anyone who has served in the Reclamation Service, have been entirely ignored by Mr. Field in his article criticizing his former associates. He has preferred to account for alleged high costs by charging "unnecessary finishing, polishing and sandpapering" and the tendency to build monuments rather than structures of necessary scope.

The only place where I ever heard contractors seriously complain of being required to do unnecessary polishing and sandpapering was on the North Platte project while it was in charge of Mr. Field. In fairness to him I will say that this policy was corrected while he was still in charge by the insertion in specifications of the following clause:

55. Finishing. All laterals may be finished with a Mormon scraper, provided that the finishing is done in a workmanlike manner and that the bottom and side slopes are even and regular, suitable for the passage of water.

The only structure I know of that can fairly be pointed out as being unnecessarily "monumental" in character is the Spring Cañon Flume, built under Mr. Field's supervision, with the approval of the consulting engineer, a view of which is given herewith. I have long regarded this as the most beautiful structure built for irrigation purposes in the United States. Its grace and majesty of design may have made it cost a little more than it otherwise would, but certainly not enough to justify any serious criticism.

The one factor of the list given by Mr. Field that actually does tend to increase the price of public over private construction is the 8-hour law, by which Government officials and contractors on Government work are prohibited from working laborers more than eight hours in any one day. This increases the cost of public work perhaps 20%; but this is not inherent in public work as such and could be remedied by the repeal of the 8-hour law.

There are also some advantages inhering in public work, the chief of which is the superior credit of the United States and the superior stability of its institutions, enabling it to secure low prices on purchases and to employ a given grade of talent cheaper than can be done by private enterprise.

It is often stated that the character of construction performed by the Government is more permanent and substantial than work for similar purposes constructed by private capital. A striking illustration of the contrast may be found on opposite banks of Grand River, Colorado, above the town of Palisade, where the left bank shows a cheap wooden flume built by a private irrigation company, which will have to be renewed in a few years, while the right bank shows the construction of a series of concrete-lined tunnels by the Reclamation Service to meet similar conditions.

ARTHUR P. DAVIS.

Washington, D. C., Nov. 14, 1913.

The Economy of Large Blast Holes in quarry blasting is generally recognized. A recent example of this as regards trap rock is afforded by the Eagle Rock Quarry Co., at East Orange, N. J. According to information furnished us, the rock is not only extremely hard, but is usually full of crevices, and as small drills tend to follow these crevices, the work with such drills is slow and costly. The average height of the quarry face is 100 ft., and the length of ledge is 1500 ft. When the drilling was done with tripod drills and the stone brought down in benches, the cost of production was very high, the face of the quarry was rough, and the plant could not be operated to its full capacity. A Cyclone drill was installed, drilling 5⅝-in. holes to the full depth of the face. Several shots have been fired and in this part of the quarry the face has been squared up and such a large quantity of stone produced as to make it possible to operate the crusher to its full capacity.

The following is the cost for a four-hole shot recently made:

335 ft. of 5⅝-in. holes	$335
1600 lb. explosive	200
Detonators and loading	15
Total cost	$550
Amount of stone dislodged	15,337 tons
Total drilling and shooting, cost per ton	3.58c.

Of the 15,337 tons, 6000 tons were so finely broken that the stone was loaded directly on the cars, without having to be run through the crusher. After the full length of the face has been shot by this method, a still greater reduction in cost will be effected, as the holes can be spaced to better advantage, for the projecting toe at the bottom of the ledge will not have to be taken care of, as is the case at the present time. In many places where the big holes have been put in, this toe has extended from 25 to 30 ft. further out than the face at the top. In spite of this unfavorable condition, the bottom stone was broken in good shape for hauling.

A Proposed New Building Code for Hartford, Conn.

A new building code for the city of Hartford, Conn., was introduced Dec. 20, 1913, into the Court of Common Council. It was ordered printed and is now under consideration for approval as law. The code was prepared by a special Commission, serving without pay, consisting of an architect, three experienced builders, an electrical contractor, the present Building Inspector, an attorney-at-law and a business man. Ford, Buck & Sheldon, Inc., Consulting Engineers, of Hartford, prepared the tentative code for discussion before the Commission and acted as advisory engineers during that discussion.

The present Building Code of the city is antique and incomplete. It contains no information on the bearing power of soils, concrete or wooden piles, skeleton-steel construction, reinforced concrete or strength of materials. It provides a Building Inspector, at $2500 per year, who shall be a competent person in the construction of brick and wooden buildings. He is appointed by the Mayor, with the approval of the Court of Common Council to serve two years.

The proposed new code is much more complete and uptodate. It creates an entirely new department called the Department of Building Inspection, which is to include the inspection of building construction, electrical and plumbing work, the plumbing having been hitherto under the direction of the Board of Health. The new Code provides for a Board of Commissioners of the Department of Building Inspection composed of five members, viz.; one architect, one general building contractor, one consulting structural engineer, one electrical contractor or electrical engineer, one plumbing contractor or sanitary engineer. Each member of the commission must have a high standing in his profession or business and have had not less than ten years' experience therein. The members of the Commission are to be appointed by the Mayor subject to confirmation by the Court of Common Council. The appointments are to be first made for the terms of 1, 2, 3, 4 and 5 years, respectively, so that the term of one member expires each year. Subsequent appointments are to be for the term of five years. No compensation for this Commission is stated.

The direct administration of the office is to be in the hands of a Supervisor of Buildings, who is also to act as Chief Building Inspector. He is to be an experienced architect, structural engineer or general building contractor and shall be appointed for a term of five years by the Board of Commissioners, after an examination. He is to receive $4000 per year. The Board of Commissioners is also to hold examinations for the appointment of all members of the Department of Building Inspection whose compensation amounts to more than $1200 per year. Those taking examinations for the office of Supervisor will be examined as to knowledge of the Building Code and its practical application, experience, and general fitness. It is provided that the principal assistants of the Supervisor shall be required to pass examinations but the Supervisor is not restricted by being compelled to accept any particular man as his assistant, provided that two or three candidates for any one examination pass, as in that event he can choose his assistant from any of the three passing the highest examinations.

It is believed that if passed the new Code will free the office of the Department of Building Inspection from politics as it will no longer be a political plum. By increasing the term of office of the Chief Building Inspector from two to five years the city should derive some benefits after a man becomes acquainted with the particular duties of the office. Under present methods a man has little more than time to learn what is expected of him in office, before it is time for his successor to be chosen. The Board of Commissioners has the power to remove, for cause, any of the appointees to office in the Department of Building Inspection. The Board of Commissioners is also made a Board of Appeals and has the power to vary the provisions of the Building Code provided four members of the Board concur in the decision. Any member of the board may veto any proposed amendment or revision of that part of the code coming under his particular field of experience.

The new Code requires drawings and specifications submitted for all structures except for "ordinary repairs." It also provides that the Supervisor may, at his discretion, require calculations to be submitted, to be made by a competent engineer, and provision is made for filing of plans for three months after completion of work.

The section on concrete and reinforced concrete is very complete and in accordance with the latest and best practice. It requires that an inspector on all reinforced-concrete buildings be employed by the owner, and report daily to the Supervisor. It also allows the use of rerolled rails, after proper test.

The area of the fire limits of the city has been greatly increased. Considerable attention has been devoted to making buildings more fireproof up to limits which will not result in uneconomical and costly construction. The new Code stipulates that all buildings five stories or more in height must be of slow burning or fireproof construction and the Code goes into considerable detail as to fireproofing. It does not allow cinder concrete.

The section of plumbing contains a new requirement in providing for filing of plans and specifications for official approval. The section covering electrical wiring is entirely new, there being no code at present. One of its important requirements is that limiting construction of installations in the fire limits to steel conduit, armor cable with metal moldings, eliminating open wiring, wood moldings and knob-and-tube work as at present allowed. The latter forms of construction mentioned are also eliminated from all churches, schools and theaters, and stables everywhere in the city. By combining the office of plumbing and electrical inspection under the Department of Building Supervision greater efficiency in all three departments is obtained, as there are many things which a member of any one department can cover and report upon even though not at present within his present jurisdiction.

Under the present ordinance, with the plumbing department separate, excavations can and have been made under foundations after the Department of Building Inspection has entirely completed its work; under the new Code this danger of undermining structures is obviated. The new Code requires that the Department of Building Supervision be notified 24 hr. in advance of placing any foundations thereby enabling the Department to pass upon the bearing capacity of the soil.

Eliminating 4% Grades on the Main Line of the Denver & Rio Grande R. R.

Grades of 4% on main lines of railways are exceptional even in mountain districts but have existed until recently on heavy-grade sections of the transcontinental routes of the Denver & Rio Grande R.R. and the Canadian Pacific Ry. On the former road a 7-mile grade of 4% has been reduced to 2% by a loop development increasing the distance to 11½ miles between the top and bottom of the grade. The similar work on the latter line was described in ENGINEERING NEWS of Nov. 10, 1910, and was of a more costly character, including spiral tunnels on the loops of the developed line. There the grade was 3.5 to 4.5% for 4 miles and was reduced to 2.2% by giving a developed length of 8.2 miles.

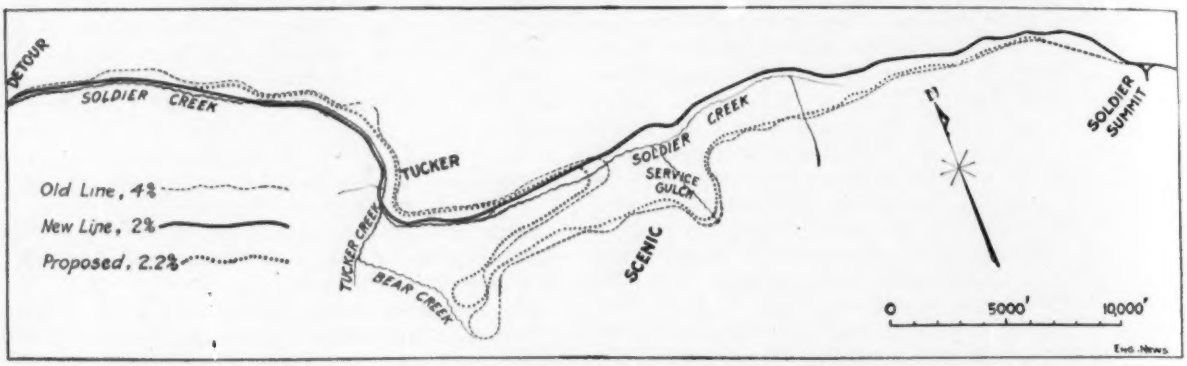
That portion of the Denver & Rio Grande R.R. constituting the through route between Denver and Ogden crosses two district mountain ranges, one in Colorado and the other in Utah. The mountain pass in Colorado lies in the main range of the Rocky Mountains and constitutes the continental divide. The range in Utah, know as the Wasatch Mountains, and intersected by the railway at Soldier Summit, constitutes the easterly rim of the Great Basin.

Prior to 1890, the Denver & Rio Grande portion of the transcontinental route was operated from Denver to Ogden as a narrow-gage line, crossing the Rocky Mountains at Marshall Pass, thence following the Gunnison River to its junction with the Grand River at Grand Junction, Colo. During 1890, the narrow-gage line extending over the Rocky Mountains by Tennessee Pass to Glenwood Springs was made standard gage, and a standard-gage connecting link was constructed from Glenwood Springs to Grand Junction to connect with the Rio Grande Western R.R., which was at that time being made standard gage.

In connection with the change to standard gage, the 4% grades of the old narrow-gage line west of Tennessee Pass were reduced to maximum grades of 3% between Tennessee Pass and Minturn, and upon completion of this work the Tennessee Pass line was made the transcontinental route. Since then, the main line operations have been conducted over 3% maximum grades descending west from Tennessee Pass to Minturn (about 30 miles), and 4% grades descending west from Soldier Summit to Tucker, Utah (27 miles). The two mountain ranges are similar in the fact that their passes are approached from the east on very much lighter grades than the descent on the west. For example, the maximum grades ascending from the east toward Tennessee Pass in Colorado are 1.42%, while the descending grades on the western slope are 3% maximum. The ascending grades from the east to Soldier Summit, in the Wasatch Mountains, are 2.4%, while the maximum descending grades west were 4% until the recent improvements were carried out.

On account of the growth of traffic, the line between Soldier Summit and Tucker was made double-track in 1900, and from time to time thereafter the second track has been extended at various critical points, so that there is now continuous double track through the Wasatch Mountains from Helper to Thistle, 50 miles.

At various times surveys have been made with a view to eliminating the 4% grades between Soldier Summit and Tucker. But the physical characteristics of this portion of the Wasatch Range, and the topography of the country adjacent to the present line, seemed to impose unusual difficulties for the development of a line on the westerly slope to such an extent as would permit material reduction in the maximum grades.



OLD AND NEW LOCATION OF THE DENVER & RIO GRANDE R.R., AT SOLDIER SUMMIT, COLO.
(Old line, 4% grade, 7 miles. New line, 2% grade, 11½ miles.)

In 1909, however, extensive additional surveys were made and the problem was studied carefully with the result of the adoption of a 2% maximum grade descending west from Soldier Summit. These surveys showed that by taking advantage of a lighter grade west of Tucker and constructing a new line 15 miles in length, the seven miles of 4% grade could be eliminated by the addition of only 4.46 miles in distance. The accompanying map shows the old 4% line, the new 2% line and a third location with a maximum grade of 2.2%.

THE NEW LOW-GRADE LINE

As might be expected, the detour line, on account of the necessary development of distance, contains a much larger amount of curvature than the old line. The maximum rate of curve is 9°, but only in a few instances was it necessary to employ this maximum rate, such cases being usually on the loops. In general the detour line is on 2% supported grade (compensated for curvature), but at passing tracks this grade is flattened to give increased facility in operation.

Owing to the fact that the line throughout its whole length lies close to the summit of the range, only comparatively small openings are necessary to provide ample waterway for both normal and flood flow of streams. In

fact, the largest opening is a 12-ft. concrete arch. All stream crossings were effected by the use of some form of concrete culvert.

The line is double track, with tracks spread 15 ft. c. to c., requiring a roadbed width of 35 ft. It is laid with the new 90-lb. rail section of the D. & R. G. R.R., which was rolled for the first time in 1913. Tie-plates are placed on every tie, and a liberal number of anti-creepers is applied. The track is ballasted with 10 in. of gravel.

Construction Work

The contract for the grading and masonry on the entire line was awarded to the Utah Construction Co., of Ogden, Utah, and work was started in January, 1913. Great difficulties were encountered by the contractors in getting equipment on the line, on account of the mountainous character of the country and of the wide divergence of the new from the old line, at various places. At some points, temporary tracks were laid to the new line and the steam shovels were moved under their own power over these tracks. At other points, steep inclines were constructed and the steam shovels were hauled up the inclines by cables.

These difficulties were considerably augmented by the requirement that entire construction must be effected in a few months and the line placed in operation in the fall of 1913. This necessitated an abnormally large amount of equipment, force reports showing that at one time 20 steam shovels were at work. Some of these, in addition to working day shifts, were required to work night shifts for weeks at a time. Generally speaking, most of the work of grading was performed by steam shovels and contractor's trains. The grading quantities moved consisted of approximately 3,500,000 cu.yd. of material, about 33% of which was solid rock, and the masonry work comprised the placing of about 13,400 cu.yd. of mass concrete.

Traffic

Freight traffic on this portion of the line consists principally of coal and coke produced in Emery and Carbon Counties, Utah, this being collected by various branches and spurs from the main line on the easterly slope of the Wasatch Mountains. The loaded movement is almost exclusively westward, the empty cars being hauled eastward to supply the demand for loading. Although the old 4% grades descended in the direction of the loaded trains they had the effect of limiting the capacity of the line, for the reason that the tonnage per train down the grade was limited by the capacity of the air pumps on the locomotives. Passenger traffic consists of three regular heavy through passenger trains each way per day. During the summer months the passenger traffic necessitates the operation of additional sections of these trains.

It is evident that the operation of the 2% line will effect great economies in operating expense, notwithstanding the fact that the new line is about 4½ miles longer than the old one. The work was under the supervision of J. G. Gwyn, Chief Engineer of the Denver & Rio Grande R.R., and Arthur Ridgway, Assistant Chief Engineer. L. B. Furman, Construction Engineer, was in direct charge of location and construction.

Automobile Engineering as a career is to be presented to students in various engineering schools in a series of talks by members of the Metropolitan Section of the Society of Automobile Engineers.

Proposed Joint Garbage and Rubbish Incinerator and Sewage Pumping Plant, Trenton, N. J.

A garbage and rubbish incinerator of 120 tons daily capacity, including just enough ashes to assist combustion and produce a good clinker, has been recommended for the city of Trenton, N. J., in report to the City Commissioners submitted on Dec. 22, 1913, by Hering & Gregory, consulting engineers, New York City. It is advised that the heat from the proposed high-temperature incinerator be converted into electric current and transmitted to the sewage-pumping station for use in lifting the sewage of the city to the new sewage-treatment works. It is also proposed that the clinker from the incinerator be used for concrete for pavement foundations. It is assumed that the incinerating plant would serve 150,000 people, or the estimated population of 1930 (the population in 1910 was 96,815). The estimated first cost of the joint plant is $368,000, of which $195,500 is for the incinerator and $172,500 for the pumping plant. The joint operating expenses, interest and sinking-fund charges are estimated at $47,310, of which $28,340 is for incinerator and $18,970 for pumping.

At present the garbage of Trenton is drained by the householders and wrapped in paper before being placed in a receptacle for collection, in accordance with a city ordinance approved Mar. 29, 1913. The garbage and rubbish are collected together by the city and burned in a Davis crematory, built in or about 1901. During the year ending Feb. 28, 1913, a total of 13,849 tons of garbage and rubbish were burned in the furnace, or about 45 tons per day on a six-days-per-week basis. More than 60 tons were sometimes burned in one day. The amount of auxiliary fuel (coal) used during the year was 275 tons. The labor force included four men, besides a superintendent and clerk at the plant. The total cost of labor, fuel, repairs, light, ash removal (from the furnaces) and miscellaneous items for the year was $6634, or 48c. per ton—"a comparatively low figure," the report states. It will be noticed that the items named do not include capital charges. At the time the furnace was visited by the engineers making the report under review no objectionable features were noted and they were informed there has been no nuisance to near-by houses since the garbage was wrapped.

The weight of garbage and rubbish collected in Trenton was 13,800 tons in 1912, and about the same in 1911. The monthly average for 1912 was 1150 tons, ranging from 693 tons in February (29 tons per day) to 1604 tons in September (62 tons per day). In 1911, also, February and September were the low and high months. The total per capita weight of garbage and rubbish in Trenton is 270 lb. (0.135 ton) per year. Separate figures not being available the engineers estimate 180 lb. (0.09 ton) of garbage and 90 lb. (0.045 ton) of rubbish per capita per year.

A total of 38,285 tons of ashes were collected in Trenton in 1912 (3190, average; 5105, maximum; 1650, minimum per month). The ashes averaged 730 lb. (0.365 tons per capita per year, making the combined garbage, rubbish and ashes 1000 lb. per year. On this basis a population of 150,000 would yield 13,500

tons of garbage, 6750 tons of rubbish and 54,750 tons of ashes per year, or a total of 75,000 tons.

In the estimates for an incinerating plant it was assumed that "not less than 20% of the total mixed refuse charged into the furnaces should be ashes," and that the remainder of the ashes should be used for filling low land, which is the present method of ashes disposal. The maximum daily quantity of mixed refuse to be burned when a population of 150,000 was reached would be 123 tons, so that two 60-ton furnaces would be ample. The estimates include three 200-hp. boilers—one for each incinerator and the third as an auxiliary coal-fired boiler to produce power for the sewage-pumping station on Sundays or at any other time when the incinerators were not being fired. The boiler evaporation was assumed at 1 lb. of water per pound of refuse burned. It is proposed that when the garbage is less than the average a larger amount of ashes should be delivered to the incinerating plant.

The engineers found that the city of Trenton now uses about 8000 cu.yd. of concrete per year for pavement foundations, requiring about 6750 cu.yd. of broken stone. This stone cost about $1.53 per ton on board cars at Trenton, or about $1.83 per cu.yd., assuming a weight of 2400 lb. per cu.yd. On the basis of 15% of clinker, of which 80% would be suitable for concrete, the incinerating plant would yield about 3000 tons of available clinker, or at 1200 lb. per cu.yd., about 5000 cu.yd. of clinker per year for a population of 150,000. This is considered less clinker than the yardage of broken stone used for pavement foundations in the Trenton of today, which has a population estimated at 110,000.

The estimated annual cost of garbage and rubbish incineration and sewage pumping has already been given as totaling $47,310. This is net cost, or what remains after allowing $5000 a year for the value of the clinker. The value of the heat from the incinerating plant does not appear directly as income credited to the incinerator, because the labor in the power plant at the incinerator is charged to the sewage-pumping station. The yearly cost is given in the accompanying table.

TABLE I. ESTIMATED ANNUAL COST AND INCOME FROM REFUSE INCINERATOR AND ESTIMETED ANNUAL COST OF SEWAGE PUMPING STATION, TRENTON, N. J.

INCINERATOR			
Operation:			
Supervision and labor	$13,680		
Removal of ashes	600		
Maintenance and repairs	1,700		
Crushing and screening clinker	2,000		
		$17,980	
Income from clinker		—5,000	
		$12,980	
Fixed charges (on $195,500):			
Interest at 3.5%	$8,800		
Sinking fund, 3.356%	6,560		
		$15,360	$28,340
SEWAGE PUMPING STATION			
Operation:			
Labor in power plant at incinerator	$2,520		
Labor in sewage pumping station	3,720		
Coal for Sunday operation	1,140		
Oil waste, packing, etc.	650		
		$8,130	
Fixed charges (on $172,500):			
Interest at 4.5%	$7,760		
Sinking fund, 1.783%	3,080		
		$10,840	$18,970
Total for joint works			$47,310

The possibility of using storage-battery electric-motor trucks, charged from the generating plant, is considered, but is dismissed for the present in view of the use the city has for all the current at the pumping station.

Estimates for garbage disposal by reduction, retaining the old crematory for burning garbage, are included in the report. This plan would involve three collections (garbage, rubbish and ashes separately), besides the abandonment of the garbage-wrapping practice just established and the provision (the report states) of tight-metal wagons for the collection of the wet garbage. It would also require extra garbage transportation. The estimates show an *extra cost of reduction over incineration* as follows: construction account, refuse disposal, $46,000; pumping station, $2300; total, $48,300. Yearly Cost: refuse disposal, $16,440; sewage pumping, $9390; total, $25,830. It should be added that the first cost of the reduction plant proper is estimated as somewhat less than the first cost of the proposed incineration plant with the electric-generating plant, but that the reduction plant would require an extra outlay of $31,500 for equipment for the separate collection of garbage and rubbish.

Failure of 8-Ft. Reinforced-Concrete Water Conduit at Montreal, Canada; City's Water Cut Off

On Dec. 25, 1913, the five-mile reinforced-concrete water conduit of Montreal burst, depriving the city of water. The break is near midlength of the conduit, where canal-enlargement excavation comes close to the pipe and below its invert level. The trouble developed during a week, more or less, beginning with signs of leakage, or seepage, which led up to the breaking out of the lower quarter of the pipe on the canal side for a length of 60 ft. on Christmas day. Temporary repairs are nearly completed as we go to press. A scanty domestic supply was provided by (1) feeding the city mains from the supply network of the Montreal Water & Power Co.* through existing connections of small capacity, and (2) distributing water with carts. The fire-protection was reduced to near zero.

The repairs made are of emergency character, but as the city has no duplicate supply-line the patched-up conduit may have to be depended upon for the year or more before flow in the old intake canal can be restored and the city's filter plant completed; the latter has been under construction for two years.

The full facts concerning the failure will not be available until the promised local investigations are made. We base the following brief preliminary account on an inspection of the break by an editor of ENGINEERING NEWS and statements to him by Water Department officials.

CANAL AND CONDUIT—The conduit, built five years ago, is 27,300 ft. long, and lies along the north bank of the open "City Aqueduct" Canal, now dry, which led from the St. Lawrence River above Lachine Rapids to the pumping station in the western part of Montreal. Its intake is formed by two 6-ft. steel pipes on the river bed extending 1200 ft. out. The conduit was planned to be the permanent supply line, and to permit emptying the canal and enlarging it for power purposes.

This enlargement of the canal has been in progress during the last few years, but in two stages (because of change of project), the second now under contract to the Cook Construction Co., Ltd. The present or second

*Supplies Westmount and several other suburbs, and a few small districts within the city itself.

enlargement (to water-line width 174 ft., depth 18 ft., and revetted or retained sides on ¼:1 batter) approaches close to the conduit (17 ft. minimum), although the latter had been built quite a distance from the old 40-ft. canal.

The conduit is 9x7½ ft., of horseshoe shape, laid on a grade of about 1:5000, with crown 4 to 5 ft. below ground surface. It carries water under a slight head (hydraulic grade 5 to 10 ft. above top of pipe). It was designed under the Department of Public Works (George Janin, Chief Engineer), in the office of T. W. Lesage, Superintendent and Engineer of the Water Department, and was built by Patrick McGovern, of Boston, Mass. Its section may be described as follows:

Invert: Radius, 13 ft.; thickness, 8 in.

Sides and arch: Inner face a part-circle of 4½-ft. radius, and so placed as to give a clear height of crown above invert 7 ft. 8½ in.; outer face, such as to give thickness at crown 8 in., thickness at base of sides 8 in., and thickness on horizontal center about 12 in.

Reinforcement: Clinton wire mesh, of size and location not now known to us.

THE BREAK—Opposite the Verdun Insane Asylum, about halfway from intake to pumping station, the canal enlargement is in clay, and approaches the conduit closely at a point where the conduit makes a slight bend concave to the canal. Finished canal bottom is 5 ft. below conduit invert. The contractor has a Bucyrus drag scraper at work, and has brought the canal section to practically full width and depth. The scraper operates on the north bank, not far from directly over the conduit, and works longitudinally. Some 5 ft. of spoil has been piled on the bank over the conduit in addition to the original cover, making at present about 10 ft. depth of earth over crown.

Water department officials state that nearly a week before the break the conduit was laid bare and a small flow of water came from a "slight fissure" in the exposed lower corner of the conduit. Probably this flow contributed to exposing the pipe; the bank just east and west of the break shows a smooth slope whose diagonal distance from the pipe is 8-12 ft. Attempts were made to close the leak by calking, and (by dumping stone behind sheeting or cribbing) to prevent wash. Canal excavation was stopped.

Wash and undermining of the pipe evidently progressed, however. The officials also mention the continuous distorting pressure caused by the surcharged bank above and landward of the conduit, and the canal side exposure. At any rate, about 5 p.m., Dec. 25, the lower 2 ft. of the southerly conduit wall, with a piece of the bottom, broke out on a length of 60 ft., and emptied the conduit. The headgates were then shut.

REPAIR—It was promptly decided to repair the break by inserting a thimble of steel pipe. A few hours after the break the Caledonian Iron Works, of Montreal, were given a rush order for 7-ft. riveted steel pipe of ¼-in. plate. The pipe has been made in 18-ft. lengths with a ring angle at each end as a bolting flange; 7 lengths were furnished, ultimately.

Excavating to clear the conduit, and cutting out the broken portion, occupied the next two days. To break the concrete mudcapping was tried, but better results were had by using the drag-line bucket as a maul. The final gap in the conduit was 62 ft. long. The steel pipe being about 125 ft. long, a lap of 30 ft. is obtained. The steel pipe was placed and cribbed up on Monday, Dec. 29, and the ends sealed into the conduit with cement mortar. It was expected that, after stiffening the exposed part of the steel pipe with timbers, water could be turned in Tuesday, Dec. 30, but Tuesday noon service had not yet been restored.

The extent of the tie-up of the water system has developed intense public indignation, promising to lead to various investigations of the whole scheme of water-supply improvement which is in progress.

The city executive department has named J. A. Jamieson and P. W. St. George a committee to report at once on the condition of the conduit. Temperatures well below freezing have prevailed since Dec. 26, so that it is supposed that in any event the conduit is secure against breakout until the Spring thaw.

Valuation of Toronto Electric Railways

The valuation of electric-railway property in Toronto, Ont., a step previous to the contemplated purchase by the municipality, has been completed by B. J. Arnold and J. W. Moyes. The task was comparatively simple, consisting of (1) finding the cost of replacement and (2) estimating the intangible values, largely those of expected profits under unexpired franchises. The properties studied were the Toronto Ry., the Toronto and York Radial Ry., and the Toronto Suburban Ry. (the last two only so far as within the city limits). An inventory and examination were made to which were applied unit prices, checked by company and city purchases and by manufacturers' quotations. The percentages added for organization, engineering and incidentals varied from 5 to 15%, averaging 6.4%. A further 3% was allowed for carrying charges during construction and 2% for legal and financing expenses. No allowance was made to cover development expense and promotion profit. Various rates of depreciation were applied to the property items; stores, real estate, legal expenses, carrying charges during construction and incidentals were not considered depreciated. The total estimate of tangible property amounted to $9,894,483 for the first company, $91,604 for the second and $51,144 for the third.

Under the existing franchise the first-named company may operate until September, 1921, the city doing all paving and furnishing track foundations. The company pays the city $800 per mile a year plus a percentage of gross receipts. Mr. Arnold decreased these last by all such payments and the usual operating expenses, leaving the rest as the profit of the company. The present value of this sum was then found for each year of the remaining franchise term and summed to give the intangible value of the properties. The appraisers found the very favorable operating ratio of 55% (operating expenses to income) applying to probable future earnings. The increase of earnings, based on probable square of population increase, was found to be 10% for five years and 8% for the remainder of the term. However, due to the probable construction of a subway in event of non-purchase by the city, the earnings were increased only 10% for three years, 8% for three years and 6% for the rest of the term. The apparent franchise value was reported as $15,701,106, but this figure was subject to a further reduction; the added investment per year to care for the traffic was estimated at $887,500, which, after

tons of garbage, 6750 tons of rubbish and 54,750 tons of ashes per year, or a total of 75,000 tons.

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News and statements to him by Water Department officials.

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Excavating to clear the conduit, and cutting out the broken portion, occupied the next two days. To break the concrete mudcapping was tried, but better results were had by using the drag-line bucket as a maul. The final gap in the conduit was 62 ft. long. The steel pipe being about 125 ft. long, a lap of 30 ft. is obtained. The steel pipe was placed and cribbed up on Monday, Dec. 29, and the ends sealed into the conduit with cement mortar. It was expected that, after stiffening the exposed part of the steel pipe with timbers, water could be turned in Tuesday, Dec. 30, but Tuesday noon service had not yet been restored.

The extent of the tie-up of the water system has developed intense public indignation, promising to lead to various investigations of the whole scheme of water-supply improvement which is in progress.

The city executive department has named J. A. Jamieson and P. W. St. George a committee to report at once on the condition of the conduit. Temperatures well below freezing have prevailed since Dec. 26, so that it is supposed that in any event the conduit is secure against breakout until the Spring thaw.

Valuation of Toronto Electric Railways

The valuation of electric-railway property in Toronto, tep previous to the contemplated purchase by the ality, has been completed by B. J. Arnold and loyes. The task was comparatively simple, con- of (1) finding the cost of replacement and (2) ng the intangible values, largely those of ex- profits under unexpired franchises. The proper- lied were the Toronto Ry., the Toronto and York Ry., and the Toronto Suburban Ry. (the last y so far as within the city limits). An inventory amination were made to which were applied unit checked by company and city purchases and by ıcturers' quotations. The percentages added for ation, engineering and incidentals varied from 5 , averaging 6.4%. A further 3% was allowed for g charges during construction and 2% for legal nancing expenses. No allowance was made to levelopment expense and promotion profit. Various f depreciation were applied to the property items; real estate, legal expenses, carrying charges dur- nstruction and incidentals were not considered de- ed. The total estimate of tangible property ted to $9,894,483 for the first company, $91,604 e second and $51,144 for the third.

ler the existing franchise the first-named company perate until September, 1921, the city doing all g and furnishing track foundations. The com- pays the city $800 per mile a year plus a percentage ss receipts. Mr. Arnold decreased these last by all payments and the usual operating expenses, leaving the rest as the profit of the company. The present value of this sum was then found for each year of the remaining franchise term and summed to give the intangible value of the properties. The appraisers found the very favorable operating ratio of 55% (operating expenses to income) applying to probable future earnings. The increase of earnings, based on probable square of population increase, was found to be 10% for five years and 8% for the remainder of the term. However, due to the probable construction of a subway in event of non-purchase by the city, the earnings were increased only 10% for three years, 8% for three years and 6% for the rest of the term. The apparent franchise value was reported as $15,701,106, but this figure was subject to a further reduction; the added investment per year to care for the traffic was estimated at $887,500, which, after

discounting interest for each year, aggregated $4,987,553, leaving as net present value of franchises for the first company $10,713,553. The values of intangibles for the other two concerns named were reported to be $193,694 and $51,144 respectively. Therefore, the total reported purchase value of the Toronto Ry. and portions of the "radial" lines within city limits was $20,944,478.

A second report by Mr. Arnold, included in a report by John Mackay on the financing of the proposed purchase, covers the probable operation of the proposed municipal system. He estimates the Toronto population at 705,400 in 1921 when the present franchise expires. The necessary track mileage would increase in that time from 147 in 1913 to 283. Added capital for new lines, added cars and power equipment, etc., is estimated to require $13,986,000. This added to the $22,000,000 which it is proposed to pay for the lines gives $45,986,000 total investment, on which fixed charges are from $1,077,000 in 1914 to $1,626,059 in 1920. Profits on the main lines are calculated on the prevailing ratio of expense to gross income, 56%; after deducting fixed and operating charge and present city percentage levy, the aggregate to 1921 is $11,500,000, which may be applied to the amortization of franchise values ($10,958,391).

Notes From Engineering Schools

UNIVERSITY OF PITTSBURGH—The Engineering School is offering an evening course in the valuation of public utilities, beginning Jan. 5, and continuing for 20 weeks, with three sessions a week. Two periods of one hour each are given up to lectures and recitations, and one period of three hours is devoted to practice work. The subjects covered are Engineering, Law, Economics and Accounting. It is planned to study the history of certain typical industries, the design and construction of their plants, the laws under which the governments may control their operations, the economic worth of valuations, and the system of accounting best suited to companies operating under public-utility laws. It is assumed that the employment of the men entering the course requires previous engineering training, so that the instruction aims to give only the special fitting required for valuation work.

NEWS NOTES

Statements That the Cameron Septic Tank Patents are controlled by the Sterilization Co., of Newark, N. J., have been given considerably publicity of late. We are informed by the Cameron Septic Tank Co., of Chicago, that the statements are "entirely unwarranted, and further that the ownership and control of the Cameron septic process patent is vested in" the Cameron Septic Tank Co.

A Drifting Balloon Record of 1739 miles is reported to have been established on Dec. 13-17 by one Kevlen who ascended at Bitterfeld, Prussian Saxony and descended at Perm, Russia. This exceeds the claimed record of Rumpelmayer for a voyage between Paris and Kharkov, Russia, in March, 1913, and far surpasses the record of 1193 miles established by Count de la Vaulx in 1900.

Proposal for a Municipal Dam to be leased to a local power company, was approved by the voters of Cedar Rapids, Iowa, on Dec. 15, 1913, 1697 to 753—women voting. The city is to build a $125,000 concrete structure across the Cedar River to replace an old timber dam. It is to be leased to the Iowa Ry. & Light Co., for 23 years, the contract expiring with the franchise of the company. The company will pay rental equivalent to about 10% on this sum. The city streets are to be lighted at a cost of $52.50 per year per arc lamp.

A Locomotive Boiler Exploded, which belonged to the Wabash R.R., on the Erie R.R. tracks, in Buffalo, N. Y., on Dec. 25. Two men were killed.

Two 15-Ton Ladle Cars Fell into the Conemaugh River, at the Cambria Steel Works, on Dec. 15; five workmen were burned. A hot-metal train of six ladle cars was on its way to the Franklin plant when the two cars left the track and toppled over into the river.

A Brick Wall Collapsed, which formed the eastern part of the old St. James Hotel, in Duluth, Minn., on Dec. 24. According to press dispatches, the building adjacent to this wall had been taken down and the wall of the hotel (probably insufficiently shored) fell into the excavation.

A 65-Ft. Telephone Pole Fell on the trolley wire along Fulton St., near Union Ave., Jamaica, L. I., on Dec. 23. According to press reports, the pole was badly rotted at the butt and had been abandoned by the New York Telephone Co., and only the alarm wires of the New York Fire Department were carried upon it.

A Heavy Sea washed out a 75-ft. section of an unfinished concrete pier at Venice, Calif., on Dec. 13. According to press dispatches, the day was clear and the ocean beyond the line of surf did not appear unusually rough. The sea was as heavy off the other coast towns in the vicinity of Los Angeles, Calif.

A Section of a Wooden Railroad Trestle Collapsed, at Augusta, Ga., on Dec. 22, due to a collision between two locomotives and 20 freight cars (the cars were standing) on the trestle. The locomotive and four freight cars fell into the Savannah River below. According to press dispatches, the engineer of one of the driving locomotives stated that there were no lights on the cars which were standing on the trestle.

The Portsmouth (England) Dockyard Burned, on Dec. 20, with an estimated loss of about $1,000,000. The fire, of unknown origin, was first discovered in a sail loft. Two lives were lost and the old semaphore tower, which in pretelegraph days was the signaling station between the fleet and the Admiralty, was destroyed. The new battle cruiser "Queen Mary," moored by the jetty, was towed beyond the danger zone.

A Panic Due to a False Alarm of Fire at a Christmas Eve celebration in Italian Hall, Calumet, Mich., caused the death of 75 persons, of whom 56 were children, on Dec. 24, 1913. About three-fourths of the 700 persons in the hall were children, and these latter were standing in the aisles, about to move forward to receive gifts from a Christmas tree, when the false alarm was given. The principal exit was a narrow stairway at the back of the hall and this was filled to the top with bodies. There was a fire escape at the rear of the building, but only those in its immediate vicinity could make use of it; others jumped from the windows which were but a few feet from the ground. The celebration was attended by several hundred striking copper miners, their wives and children.

An Elevated Railway Rapid Transit System for Chicago, consisting of a development of the present elevated railway system, is being advocated by the Northwest Side Commercial Association. At present there are four lines entering at the loop in the business center of the city, and these have a number of branches, while the traffic of all the lines converges to and passes through the congested business district. The first improvement suggested consists in the construction of about 7½ miles of line to link up existing lines and create a north-and-south elevated line just west of Ashland Ave., enabling persons to pass between the northern and southern sections of the city without passing through the loop district. Numerous future extensions of the existing lines into the outlying sections are projected also, as well as an elevated line just east of Halsted St. (in a densely populated factory neighborhood) from from North Ave. to 63d St.

The through routing of elevated trains, recently put into effect, has somewhat reduced the congestion on the loop, but still practically every train on the elevated railways has to pass through the loop district, in spite of the fact that the westward growth of the city has developed districts which are quite independent of the business center of the city. It is estimated that linking up the existing lines to form a through route as above described would greatly reduce the congestion on the elevated railways.

Dissolution of the Alliance between the Bell telephone and Western Union telegraph systems has been agreed upon between these companies and the U. S. Attorney General, to prevent suits under the Sherman Act. The American Tele-

phone & Telegraph Co. also has agreed not to acquire any competing systems and to extend toll-line service to other telephone companies. This separation will halt the expected unification of the Bell and Western Union services and the further economies which were expected to result from a more complete merging of equipment than had been secured.

Steel Rails and Other Track Equipment for the new New York City Subway System are to be bought under contract with separate manufacturers by the Public Service Commission. Formerly the practice was to let contracts for the work and allow the contractor to purchase the materials. Under the new plan the Commission will advertise for bids for the supply of such materials, and after awarding the contracts will make another contract with contractors experienced in building railroads to do the work of track laying, etc. Of the 324 miles of single-track construction called for by the Dual System contracts, the work on city-owned lines will cover 260 miles. As the specifications call for steel rails weighing 100 lb. to the yard, it is calculated that the Commission will purchase about 520 miles of single rails, which will weigh about 45,760 tons. On account of sidings, etc., it is probable that the actual amount will exceed these figures. A public hearing on the form of contract for supplying such materials will be held by the Commission on Jan. 9, 1914.

PERSONALS

Mr. W. F. Drysdale, recently Mechanical Engineer of the Northern Ry., of Costa Rica, at Limon, Costa Rica, has resigned.

Mr. Perry T. Hanscom, Assoc. Am. Inst. E. E., Assistant to the President of the Great Western Power Co., San Francisco, Calif., has resigned.

Mr. R. L. Cooper, recently Engineer of the Conn Construction Co., has been appointed Chief Engineer of the Fort Dodge, Des Moines & Southern R.R., at Boone, Iowa.

Mr. John J. Mantell, recently Terminal Trainmaster of the Erie R.R., at Jersey City, N. J., has been promoted to be Superintendent of the Wyoming division at Dunmore, Penn.

Mr. T. E. Coyle, recently Assistant Superintendent of the Northern Pacific Ry., at Tacoma, Wash., has been promoted to be Superintendent of the Pasco division, at Pasco, Wash.

Mr. Wm. Neland, formerly Chief Inspector of the Board of Public Works of Honolulu, H. I., is now Superintendent of Construction for the Kauffman-Price Construction Co., Oakland, Calif.

Mr. A. L. Dabney, M. Am. Soc. C. E., Chief Engineer of the Dabney Engineering Co., Memphis, Tenn., has been selected to make plans for a proposed new water-works system for Vicksburg, Miss.

Mr. L. S. Rose, Signal Engineer of the Cleveland, Cincinnati, Chicago & St. Louis R.R., has been appointed Engineer in charge of valuation work, with headquarters at Cincinnati, Ohio.

Mr. Augustine W. Wright, M. Am. Soc. C. E., whose resignation as a member of the Board of Public Utilities of Los Angeles, Calif., was noted in our personal columns of Dec. 18, has withdrawn his resignation at the special request of Mayor Rose, and has been elected President of the Board.

Mr S. E. Neil has been appointed County Highway Engineer of Marion County, Ala. He has previously been connected with the Tennessee Coal, Iron & Railroad Co., the American Coal Corporation, the engineering department of the city of Birmingham, Ala., and Milner & Browne, Civil Engineers, Birmingham.

Mr. John R. Brownell has resigned as Safety Engineer of the Pennsylvania Steel Co., Steelton, Penn., to become Superintendent of Safety of the State of California, with headquarters at Sacramento. This is a newly created position, the duties of which are similar to those of the Commissioners of Labor of some of our Eastern states.

Mr. Milo R. Maltbie, Member of the Public Service Commission, First District, of New York State, has been appointed Chairman of the executive committee of the Committee of Fifteen, appointed by the National Association of Railway Commissioners to represent the state public service commissions in the appraisal of interstate railways.

Mr. George Hall Hazlehurst, a graduate in civil engineering of Rensselaer Polytechnic Institute and Harvard University, has resigned his position as Sanitary Engineer of the Atlantic Coast Line Ry., at Wilmington, N. C., to become associated with his father, Mr. James Nisbet Hazlehurst, M. Am. Soc. C. E., Consulting Municipal Engineer, Atlanta, Ga.

Mr. W. D. L. Hardie, a civil and mining engineer of Western Canada, has been elected Mayor and Commissioner of Finance and Safety of the city of Lethbridge, Alta., under the new commission plan of city government. The other two commissioners are Mr. A. M. Grace, M. Am. Soc. C. E., Commissioner of Public Works, and Mr. Arthur Reid, Assoc. Am. Inst. E. E., Commissioner of Public Utilities.

Mr. Frederick W. Mayes, formerly with the New York Insurance Exchange, New York City, has been made the head of the Brooklyn Engineering Insurance & Inspection Bureau, established as a special department of Pendleton & Pendleton, Brooklyn, N. Y. The purpose of the bureau is to serve in an advisory capacity to architects, builders, owners, manufacturers and others on matters of reduction of insurance rates, fire protection and prevention and all other problems related to insurance engineering.

Mr. Frank G. Baum, M. Am. Inst. E. E., Assoc. M. Am. Soc. C. E., Consulting Engineer, San Francisco, Calif., and recently Chief Engineer of the Lake Spaulding dam and the South Yuba hydro-electric power development of the Pacific Gas & Electric Co., described in our columns of Dec. 11, 1913, was the guest of honor at a banquet tendered him on Dec. 19 by the officers of the company. About 200 guests were present, including many prominent engineers and business men. Mr. John A. Britton, Vice-President and General Manager, was toastmaster.

Mr. Kingsley L. Martin, M. Am. Soc. C. E., has resigned as Vice-President of the Foundation Co., New York City, to become President of the Whiting Paper Co., Holyoke, Mass. He will make his home in Springfield, Mass. Mr. Martin was formerly Bridge Commissioner of New York City, after having been connected with the bridge department of the city for many years. He was Assistant Engineer of the Brooklyn Bridge in 1892 and Assistant Engineer of the Williamsburg Bridge in 1896. During the Spanish-American War he served in the United States Navy. In 1904 he was made Engineer in charge of the Williamsburg Bridge, and in 1908 Chief Engineer of the Department of Bridges. Mr. Martin was elected Vice-President of the Foundation Co., in June, 1910.

Mr. T. H. Beacom, Assistant General Manager of the First district of the Chicago, Rock Island & Pacific Ry., at Des Moines, Iowa, has been promoted to be General Manager of the Third district, at El Reno, Okla., succeeding Mr. Charles W. Jones, who has been promoted to be General Manager of the First district. Mr. Beacom is a native of Iowa and began his railway experience at 16 years of age as a timekeeper with a construction gang on the Chicago, Milwaukee & St. Paul Ry. His service with the Rock Island began in 1884 as a brakeman; he was promoted to be conductor, yardmaster, trainmaster, and in February, 1904, to be Superintendent of the Oklahoma division. In December, 1909, he was made General Superintendent, and in 1912 Assistant General Manager at El Reno.

Mr. C. P. Fortney, Assoc. M. Am. Soc. C. E., recently Supervisor of Machinery Erection, at the Gatun Locks, Panama Canal, has resigned to accept a position under Maj. John C. Oaks, Corps of Engineers, U. S. A., on the construction of one of the dams in the Ohio River, near Louisville, Ky. Mr. Fortney entered the service of the Isthmian Canal Commission on Aug. 22, 1907, as a draftsman on masonry and valve design. In April, 1909, he was transferred to the field in charge of certain parts of the work on the Gatun locks construction plant. The following September he was promoted to be Superintendent of Masonry Construction at Gatun. Two years later he was placed in charge of the installation of the permanent lock-operating machinery, which is the position he has just resigned, the work having been practically completed.

Mr. Frederic A. Delano, M. Am. Soc. C. E., former President and one of the receivers of the Wabash R.R., has been elected President of the Chicago, Indianapolis & Louisville Ry., succeeding Mr. Fairfax Harrison, resigned, as noted in our columns of Dec. 11. Mr. Delano is well known as a writer on transportation subjects and has had a varied and interesting experience in engineering and railway work. Born in Hong Kong, China, in 1863, he was graduated from Harvard University and entered the railway service at the age of 22 years as a machinist apprentice of the Chicago, Burlington & Quincy R.R., at Aurora, Ill. Two years later he was placed in charge of the bureau of steel rail inspection tests and records. For a year, 1889-1890, he was Assistant to the Second Vice-President, and was then appointed Superintendent of freight terminals at Chicago, Ill., a position which he held for nine years. In 1899 he was promoted to be Superintend-

ent of Motive Power. From 1901 to 1905 he was General Manager. For a few weeks in 1905 he was Consulting Engineer to the United States War Department and the Philippine Commission, after which he returned to the railway service as Vice-President of the Wabash R.R. In October, 1905, he was elected President, and since 1911 has been Receiver.

Mr. Ernest D. Bean, Assoc. M. Am. Soc. C. E., has been appointed Chief Engineer of the Palisades Interstate Park Commission of New York and New Jersey. Mr. Bean is a graduate of the University of Maine, at Orono, Me., and has been for several years superintendent of construction for the firm of Barrally & Ingersoll, contractors and engineers, which has contracts on the western section of the New York Barge Canal. Prior to this connection Mr. Bean was connected with the engineering department of the New York State barge canal work and with the State Highway Department.

Mr. Lewis E. Moore, Assoc. M. Am. Soc. C. E., Assistant Professor of civil engineering at the Massachusetts Institute of Technology, has been appointed Engineer of Bridges and Signals of the Massachusetts Public Service Commission, succeeding Dr. George F. Swain, President, Am. Soc. C. E., resigned as noted in our issue of Dec. 11. Mr. Moore graduated in mechanical engineering at the University of Wisconsin in 1900. The subsequent year he spent at the Massachusetts Institute of Technology studying structural engineering. One of his first engineering experiences was as Master Mechanic of an iron mine in upper Michigan. Subsequently he was connected with the Phœnix Bridge Co., the Bucyrus Steam Shovel Co., and the Western Electric Co. He was also employed at various times on the Illinois Central R.R., of which his father was at one time Chief Engineer. Mr. Moore taught for a year at the University of Wisconsin and three years at the University of Illinois before going to the Massachusetts Institute of Technology in 1907 to take charge of the course in bridge design. Since his connection with the faculty of the Institute he has also been employed by the Massachusetts Railroad Commission and its successor, the Public Service Commission, in the inspection of and computation of stresses in bridges; besides which he has had an independent consulting practice.

Mr. Howard S. Reed, Assoc. M. Am. Soc. C. E., whose resignation as Engineer, United States Reclamation Service, was noted in our issue of Sept. 11, has entered private practice as Consulting Engineer, 215 Noll Bldg., Phœnix, Ariz. Mr. Reed began engineering work in 1894, and for three years he was connected with Moore & Co., of Boston, Mass., in the building of water-works in various parts of New England. In 1897 he received an appointment in the employ of the Nicaraguan Canal Commission and was assigned to a hydrographic party under Mr. Arthur P. Davis, M. Am. Soc. C. E., now Chief Engineer of the United States Reclamation Service. Mr. Reed continued in the employ of the Nicaraguan Canal Commission and its successor, the Isthmian Canal Commission, until 1902, when the work was concluded. Returning to Washington, D. C., he was assigned to the United States Geological Survey and in June, 1902, was placed in charge of hydrographic studies on the Vinta Indian Reservation in northeastern Utah. Upon the completion of this work he was transferred to the Salt River Project of the United States Reclamation Service, and for some time was engaged in general engineering and various hydrographic investigations in Arizona. In May, 1907, Mr. Reed was placed in charge of operation and maintenance of the project, where he remained until the time of his resignation. In 1908 he designed and made plans and specifications for the installation of a water-works system for the city of Phœnix, and has since acted in a consulting capacity for the city in the purchase of pumps and in various engineering matters connected with an investigation of a gravity water-supply for the city.

OBITUARY

Lester Adamson Dolley, for several years engaged in railway engineering and mining work in Mexico, died in Mexico City, Dec. 8. He was a native of Rochester, N. Y.

James G. Hill, F. Am. Inst. Arch., former Supervising Architect of the United States Treasury Department, died at his home in Washington, D. C., Dec. 20, aged 72 years.

James D. Kilpatrick, President of the McGraw Construction Co., Philadelphia, Penn., died Dec. 24. He was 45 years old and the son of Samuel Kilpatrick, who was also a prominent contractor.

William D. Platt, member of the Baltimore, Md., Sewerage Commission, died Dec. 23. He was one of the members of the original commission appointed in 1905 and had served ever since.

Arthur Barnes Treat, head of the contracting firm of Sperry & Treat Co., New Haven, Conn., died Dec. 26, from acute indigestion. Mr. Treat's firm was contractor for numerous school and college buildings throughout the East. Many of the most notable buildings at Yale University were constructed by him.

Hoyt H. Green, President of the Green Car Wheel Co., St. Louis, Mo., died Dec. 19, at his home in St. Louis. He was born in Massachusetts in 1836. He went to St. Louis from the Mowry Car Wheel Co., of Cincinnati, Ohio, in 1856, and with his brother, D. P. Green, founded the Green Car Wheel Co. He was also President of the American Hardwood Lumber Co., and Vice-President of the Williams Patent Crusher & Pulverizing Co., of St. Louis.

ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN ASSOCIATION FOR ADVANCEMENT OF SCIENCE.
Dec. 29-Jan. 3. Sixty-fifth annual meeting, at Atlanta, Ga. Secy., L. O. Howard, Smithsonian Institution, Washington, D. C.

SOCIETY OF CONSTRUCTORS OF FEDERAL BUILDINGS.
Jan. 5-7. Fifth annual convention at Washington, D. C. Secy., T. R. Maul, 409 P. O. Building, Philadelphia, Penn.

MINING AND METALLURGICAL SOCIETY OF AMERICA.
Jan. 13. Annual meeting at New York City. Secy., W. R. Ingalls, 505 Pearl St., New York.

AMERICAN SOCIETY OF ENGINEERING CONTRACTORS.
Jan. 16. Annual convention in New York. Secy., J. R. Wemlinger, 13 Park Row, New York City.

AMERICAN WOOD PRESERVERS' ASSOCIATION.
Jan. 20-22. Tenth annual convention at New Orleans, La. Secy., F. J. Angier, Timber Preservation Co., Baltimore, Md.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Jan. 21 Annual meeting in New York. Secy., Chas. W. Hunt, 220 W. 57th St., New York City.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
Jan. 21-23. Annual meeting in New York. Secy., E. A. Scott, 29 W. 39th St., New York City.

CANADIAN SOCIETY OF CIVIL ENGINEERS.
Jan. 27-29. Annual meeting in Montreal, Que. Secy., Prof. C. H. McLeod, 176 Mansfield St., Montreal, Que.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Jan. 29-31. Midyear meeting in New York. Secy., E. B. Burritt, 29 W. 39th St., New York City.

NATIONAL LIME MANUFACTURERS' ASSOCIATION.
Feb. 4-5. Annual meeting at New York. Secy., Fred K. Irvine, Chicago, Ill.

NATIONAL CONFERENCE ON CONCRETE ROAD BUILDING.
Feb. 12-14. Meeting at Chicago, Ill. Secy., J. P. Beck, 72 W. Adams St., Chicago, Ill.

AMERICAN CONCRETE INSTITUTE
Feb. 16-20. Annual meeting in Chicago, Ill. Secy., E. E. Krauss, Harrison Building, Philadelphia.

Kansas Engineering Society—The annual convention will be held in Lawrence, Kan., Jan. 20-21. The Secretary is T. J. Stricklen, Topeka, Kan.

International Electrical Congress—The Congress is to be held at San Francisco, Calif., Sept. 13-18, 1915, under the auspices of the American Institute of Electrical Engineers, by the authority of the International Electrotechnical Commission, and during the Panama-Pacific International Exposition. There will probably be about 250 papers presented. The first membership invitations will be issued by March, 1914. The meeting of the International Electrotechnical Commission will be held during the week preceding that of the Electrical Congress, while during the week immediately following the Congress the International Engineering Congress will be held.

American Institute of Electrical Engineers—The second annual midwinter convention will be held in the Engineering Societies Building, at 29 W. 39th St., New York City, Feb. 25-27, under the auspices of the Electric Power Committee. The general subject of the meeting will be "electric power," and each of the subcommittees will present papers on the special branch under its jurisdiction. The subdivisions of the subject are: (1) power stations; (2) power generation; (3) protective apparatus; (4) transmission; (5) distribution; (6) economics, and (7) engineering data. A full list of the papers to be presented at this meeting is not available at this time; but a complete program of the convention will be published in the February issue of the "Proceedings."